STORM DRAINAGE REPORT

BELLEVUE CHILDREN'S ACADEMY - Willows Preparatory School Expansion Project A REDMOND, WA

For SPE Approval March 2017

Prepared For:

Bellevue Children's Academy

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1.1 Project Overview

The Willows Preparatory School Project A is part of the Bellevue Children's Academy Master Planned Development (BCAMPD) of the Willows Preparatory School. The Willows Prep Project A proposes to redevelop approximately 1.17 acres of the property located in Section 26, Township 26 North, Range 5 East, Willamette Meridian in the City of Redmond. The entire site area is approximately 12.80 acres in size and is located south of NE 124th Street and east of Redmond-Woodinville Road NE (SR 202). See Figure 1 for the site location map and Figure 1.2 for the aerial photograph. The project site is recorded with tax parcel number 2626059071, in King County, Washington.

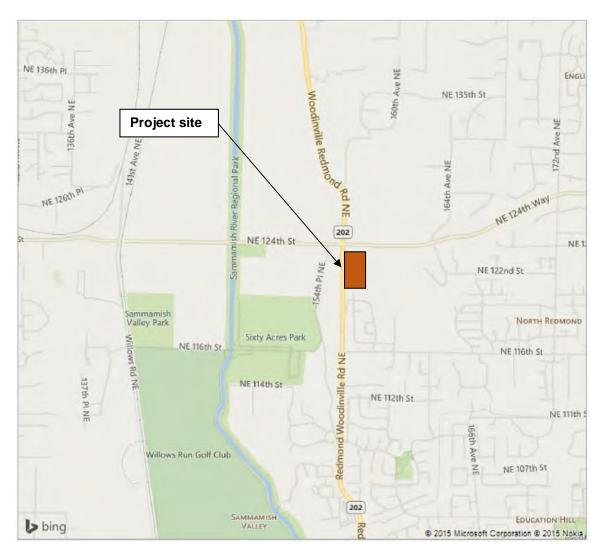


Figure 1: Site Location Map





Figure 1.2: Aerial Photograph

The project site is zoned as "R-1, R-4". Its current use is church/religious services under a conditional use permit.

The Willows Prep Project A proposes to remove several existing structures and construct private school classrooms for approximately 300 middle-school age students. There is a complex of 6 existing buildings on the subject property. The two larger buildings are currently in use by a church group (Washington Cathedral), renting from Willows Prep, and will remain undisturbed. The proposed Willows Prep Project A will include two 2-story classrooms for 300 students total, a covered patio and an uncovered plaza over a new single-story structure.

The site development engineering for Project A includes site grading, and storm drainage improvements for the site. The storm drainage improvements will include installation of catch basins and underground storm pipe system to collect and convey stormwater to the existing storm system in Redmond-Woodinville Road NE.



SECTION 2 – MINIMUM REQUIREMENTS

The Willows Prep Project A will follow the City of Redmond stormwater standards. The project entered the PREP process after March 1, 2016 and has completed the 60% intake meeting before December 31, 2016 therefore the project is subject to the 2016 standard. The 2016 standard allows for the use of the 2012 Washington State Department of Ecology Stormwater Management Manual as amended in December 2014 (2014 SWMMWW), with modifications contained in the City's Stormwater Technical Notebook. The flow chart for determining requirements for re-development was used to assess the minimum requirements for the proposed project. This project is required to meet Minimum Requirements #1- #5 in Chapter 2 of the Stormwater Technical Notebook. See the flow charts for the redevelopment on the following pages.



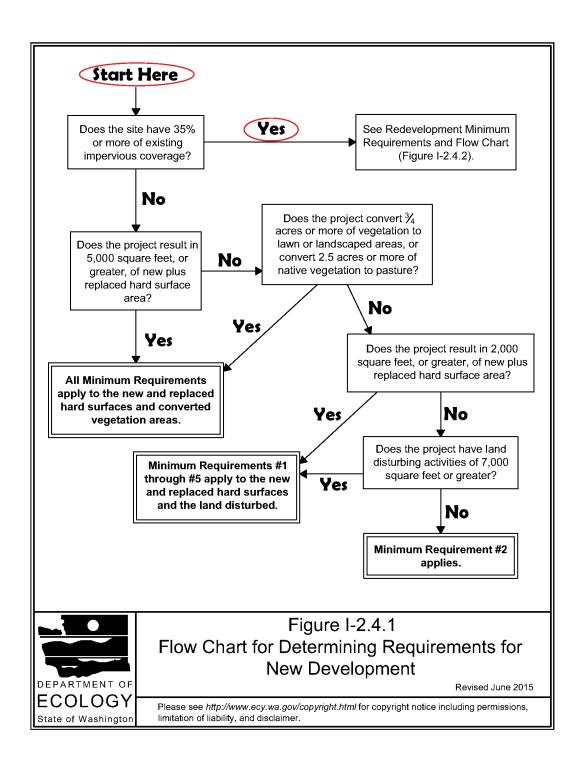


Figure 2.1: Flow Chart for Determining Requirements for New Development



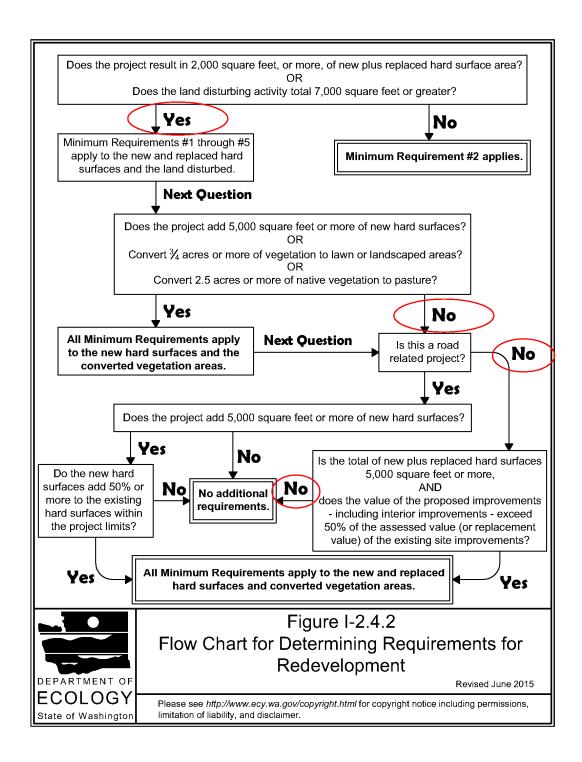


Figure 2.2: Flow Chart for Determining Requirements for Redevelopment



2.1 <u>Minimum Requirement #1</u> Preparation of Stormwater Site Plans

A Stormwater Site Plan has been prepared for the proposed Willows Preparatory School expansion per the 2014 SWMMWW and the City of Redmond Stormwater Technical Notebook. The site plans include stormwater engineering design plans and this storm drainage report, including technical design calculations.

2.2 <u>Minimum Requirement #2</u> Construction Stormwater Pollution Prevention Plan (C-SWPPP)

A Temporary Erosion and Sediment Control plan has been prepared to comply with the Construction SWPPP Elements #1 through #12. The Elements are listed in this storm drainage report under Section 5.

2.3 <u>Minimum Requirement #3</u> Source Control of Pollution

The proposed use of the project is a school where the source of potential pollution will be the parking area. The proposed buildings will be located over the area of the existing structures that are to be removed. Non-pollution generating impervious surface in the form of concrete walkways and a plaza over a proposed structure will replace a portion of the existing parking area. Asphalt parking around the structures will be improved, but the square footage of asphalt parking will be decreased from the existing conditions.

2.4 <u>Minimum Requirement #4</u> Preservation of Natural Drainage Systems and Outfalls

The portion of the site for redevelopment currently drains north and northwest to the public storm drainage system located in Redmond-Woodinville Road NE. The redevelopment will preserve the existing outfall. Stormwater will be released from the project site at the same location and continue to flow to the existing system.

2.5 <u>Minimum Requirement #5</u> On-site Stormwater Management

Per the City of Redmond requirements, the project shall employ on-site Stormwater Best Management Practices (BMPs) to infiltrate, disperse and retain stormwater runoff onsite to the maximum extent feasible without causing flooding, groundwater contamination or erosion impacts. See Section 4.5 for an evaluation of on-site BMPs. BMPs in List #1 will be evaluated in the order presented and used when feasible.



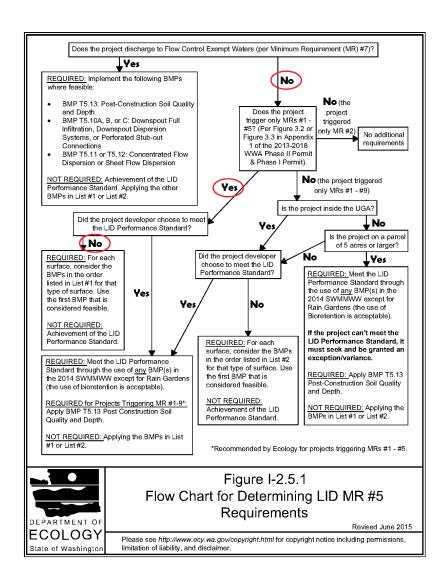


Figure 2.3: Flow Chart for Determining LID Requirements



SECTION 3 – OFF-SITE ANALYSIS

3.1 Upstream Analysis

No upstream area is tributary to the project site as all upstream flows are intercepted by conveyance pipe and catch basin systems. Stormwater runoff is directed to detention ponds, outfalling to the public storm drainage system in Redmond-Woodinville Road.

3.2 Downstream Analysis

Stormwater runoff from the site sheet flows northwesterly towards the west property line. Stormwater directly enters the public storm drainage system in Redmond-Woodinville Road NE which is located on the east gutter line of the road. Stormwater continues north in the underground public storm system for 136' through 12" piping to a catch basin just south of the intersection at NE 124th Street. Drainage exits the catch basin to the west across Redmond-Woodinville Road NE for 59' to a catch basin on the west side of the road.

Drainage continues northwest for 58' through 12" piping to a catch basin located in the south gutter line of NE 124th St. It flows west from this catch basin for 160' through 12" ADS to a tributary of the Sammamish River. The tributary meanders across King County owned parcels along the south side of NE 124th St, outfalling to a pond approximately 1800 feet downstream of the project site. The tributary continues meandering westerly, ultimately outfalling to the Sammamish River approximately 3000 feet downstream of the project site.

The study area for the project extends ¼ mile downstream of the project site discharge location. Refer to Figure 3 below for the downstream drainage paths for the basin. There are no reported drainage problems with the conveyance pipe system along the downstream path.

3.3 Critical Areas

The phase 1 site area does not contain erosion hazard areas, frequently flooded areas, landslide hazard areas, seismic hazard areas, or wetlands. Highschool Creek is located to the east of the proposed phase 1 building. Highschool Creek is a Type II stream with a 150' buffer that is partially located near the project limits. The 150' stream buffer is located approximately 200' from the proposed development area in phase 1. The stream buffer will not affect the development of the site for this project.



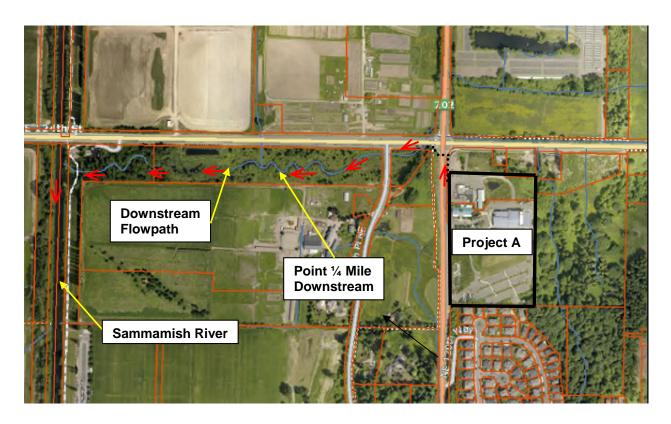


Figure 3: Downstream Drainage Map



4.1 Existing Site Hydrology

The Willows Prep Project A portion of the site, consists of several structures, concrete patio, asphalt concrete pavement used for access and parking, concrete curbing, and landscaping. Surface runoff on the existing project site sheet flows to the north and is collected in a series of catch basins located north of the parking area and conveyed to the existing stormwater system in Redmond-Woodinville Road NE. Stormwater then flows westerly in the public system, eventually outfalling to the Sammamish River as described in *Section 3 – Offsite Analysis*. A detention pond exists to the east of this phase of the project. This pond is utilized by the two larger building that are to remain. Construction of this project has been planned to protect the existing pond and will not utilize the pond to store stormwater generated by the portion of the site being developed in this project.

4.2 Developed Site Hydrology

The Willows Prep Project A proposes to redevelop approximately 1.17 acres of the 12.80 acres of the site by replacing existing structures with two classroom buildings and reconfiguring the parking, drive aisle, and landscape areas. The new structures will be placed on top of the area where the existing structures stood and the parking area will be reconfigured, minor grade changes will take place around the site to maintain positive drainage toward catch basins.

Stormwater flow from the remainder of the existing project area will not change as part of this development.

Table 1. Summary of Site/Disturbed Areas

Table 1: Cammary of Ottor Biotarbea Air			
	Area		
Existing	(square feet)		
Impervious	45,330		
Pervious (landscaping)	5,620		
Total Area	50,950		
Proposed			
Impervious	37,480		
Pervious (landscaping)	13,470		
Total Area	50,950		
Decrease Impervious	7,850		

Refer to Appendix A for existing and proposed site area maps.



4.3 Conveyance System Analysis and Design

The Willows Prep Project A conveyance system will be sized to convey the 25-year peak storm event using the Rational Method to calculate peak flows within each drainage subbasin. Conveyance calculations are included in Appendix C. The pipe network is capable of conveying 3.87cfs which is greater than the 25-year flow of 2.16cfs. A full backwater analysis will be prepared for the civil review of the project.

4.4 LID Assessment

The site has been assessed for Low Impact Development to determine the feasibility of infiltration and dispersion Best Management Practices (BMPs). List #1 has been used to evaluate On-site Stormwater Management BMPs for Project Triggering Minimum Requirements #1 through #5. On-site Stormwater Management Techniques evaluated are as follows:

1. Lawn and Landscaped Areas:

The project will amend soils within landscaped areas in accordance with the City of Redmond Standard Detail 632.

2. Roofs:

Full Dispersion has been evaluated for this site and deemed to be infeasible because the required native vegetated flow path does not exist on the site.

Full Downspout Infiltration Systems have been evaluated and deemed to be infeasible. Infiltration was evaluated and determined to not be a viable stormwater management practice as the project is located within an area of existing reconstructed parking area. There is minimal landscape areas and reductions in this area reduce the ability of emergency vehicles to maneuver. The near surface soils on-site generally consist of medium stiff silt and clay fill overlying dense to very dense silty sand with gravel. Based on the soil types provided in the geotechnical report, infiltration is not feasible.

Rain Gardens have been evaluated and deemed to be infeasible due to low infiltration rates and recommendations made by the geotechnical engineer. The site is underlain by glacial till and transitional bed. Both materials are considered close to impermeable and unfavorable for infiltration.

Downspout Dispersion Systems were evaluated and determined to not be a viable stormwater management practice due to the location of the structures and the paved parking. A vegetated flow path could not be achieved.

Perforated Stub-out Connections were evaluated and determined to not be a viable stormwater management practice as the project is located within an area of existing reconstructed parking area. The BMP requires the perforated pipe



portion of the system should not be located under impervious surfaces. There is minimal landscape areas and reductions in this area reduce the ability of emergency vehicles to maneuver. The site is underlain by materials that have very low infiltration rates. Water from the perforated by would flow through the gravel trench into the public storm system with little to no attenuation.

Other Hard Surfaces:

Full Dispersion has been evaluated for this site and deemed to be infeasible because the required native vegetated flow path does not exist on the site.

Permeable Pavements, rain gardens, and bioretention have been evaluated and deemed to be infeasible due to low infiltration rates and recommendations made by the geotechnical engineer. The site is underlain by glacial till and transitional bed. Both materials are considered close to impermeable and unfavorable for infiltration.

Sheet Flow Dispersion has been evaluated for this site and deemed to be infeasible because the required vegetated flow path does not exist on the site.



SECTION 5 – CONSTRUCTION STORMWATER POLLUTION AND PREVENTION C-SWPPP

5.1 Temporary Erosion and Sediment Control & SWPPP

This section covers Construction Stormwater Pollution and Prevention (C-SWPPP). Along with this storm drainage report, a temporary erosion and sediment control (TESC) plan has been developed for the proposed redevelopment project. The TESC plan contains standards per the 2014 SWMMWW that are put in place in order to prevent or reduce pollution of stormwater runoff caused by construction activities and to minimize the amount of sediment-laden runoff leaving the project site.

The Willows Prep Project A site will be an open excavation type construction. Catch basin inlet protection will be installed at existing catch basin that could receive sediment runoff. To reduce erosion of exposed soils during the open cut excavation, numerous temporary cover practices will be implemented per the 2014 SWMMWW. These practices include catch basin inlet protection, and filter fabric fence. The 2014 SWMMWW requires projects to address twelve elements as part of the Construction SWPPP as follows:

5.1.1 Element #1 – Mark Clearing Limits

The portion of the site for Willows Prep Project A will be surrounded by temporary metal fencing during construction.

BMP C103 - High Visibility Plastic or Metal Fence

5.1.2 Element #2 – Establish Construction Access

The existing pavement and driveway will be used for construction access. Exposed soils will be covered to prevent the movement of sediment to offsite roads. Any sediment tracked off the site will be cleaned by a street sweeping machine.

BMP C107 - Parking Area Stabilization

5.1.3 Element #3 – Control Flow Rates

During construction, stormwater generated on-site will be collected in a swale along the north property line and directed to settlement tanks. Flow control during construction will be provided by the settlement tanks. Once the site is constructed, flow control is not required for this project.

5.1.4 Element #4 – Install Sediment Controls

Sediment controls pertinent to the project shall be installed before the start of construction. Storm drain inlet protection will be installed on existing and proposed catch basins near the vicinity of the project.

BMP C220 - Storm Drain Inlet Protection



5.1.5 Element #5 - Stabilize Soils

All exposed soils shall be covered and stabilized prior to construction.

BMP C123 - Plastic Covering

5.1.6 Element #6 – Protect Slopes

Exposed slopes will be covered with temporary plastic covering or mulching to prevent erosion of soils. Surface pollution prevention will also be controlled for pavement sawcutting.

BMP C121 - Mulching

BMP C123 - Plastic Covering

BMP C152 – Sawcutting and Surface Pollution Prevention

5.1.7 Element #7 – Protect Drain Inlets

Storm drain inlet protection will be placed on all catch basin rims in the vicinity of construction to prevent sediment from entering the storm drainage system prior to permanent stabilization of the construction site.

BMP C220 – Storm Drain Inlet Protection

5.1.8 Element #8 – Stabilize Channels and Outlets

The Willows Prep Project A outlet is to an existing catch basin. The catch basin runs through an underground drainage system. The outlet at the discharge point is currently stabilized. See *Section 3 – Offsite Analysis* for more information.

5.1.9 Element #9 – Control Pollutants

Pollutants that may be generated from the project site include those from normal construction activities. These can include machinery oils and dust. The proposed project site will provide storage and containment for any possible pollutants and dust control.

BMP C140 – Dust Control

BMP C153 - Material Delivery, Storage and Containment

5.1.10 Element #10 – Control De-Watering

De-watering for the project site should not be necessary. Groundwater is not anticipated within the vicinity of the redevelopment.

5.1.11 Element #11 - Maintain BMP's

The contractor shall implement additional Best Management Practices as required and/or recommended by the City's inspector and as approved and/or required by other agencies during construction to prevent construction debris, waste, material, fuel, oil, lubricants, and other fluids from entering the public right of way and the existing storm conveyance system.



5.1.12 Element #12 – Manage the Project

A construction sequence will be developed for sequencing of erosion and sediment control installation timelines. A CESCL will be responsible for and manage TESC during construction. All construction debris will promptly be removed from the site to minimize demolition and construction impacts on the site.

5.2 Construction Schedule

Construction of the classrooms is planned to begin fall of 2017 with a finished date in summer, 2018. Construction will take place during the dry season and no additional wet weather requirements are anticipated. Normal TESC BMP practices will take place as explained in Section 5.1.

5.3 Permanent Erosion and Sediment Control

After completion of construction, the site will be permanently protected from possible erosion and sediment problems. All exposed soils shall be stabilized using permanently seeding.

BMP C120 - Temporary and Permanent Seeding

5.4 Risk Assessment

Under existing conditions, there are no known erosion problems on the site. The temporary and permanent erosion and sediment control measures described above, along with any additional BMP's as deemed necessary, will be implemented to protect downstream resources.



SECTION 6 – SPECIAL REPORTS AND STUDIES & OTHER PERMITS

6.1 Special Reports and Studies

Below is a list of reports and studies that were completed for the project and can be found in Appendix B.

- Geotechnical Engineering Services, Prepared for Washington Cathedral Prepared by GeoEngineers, Inc., dated July 26, 2004
- Geotechnical Recommendation provided by email, dated March 9, 2017

6.2 Other Permits

A Building Permit will be obtained from the City of Redmond.

SECTION 7 – OPERATIONS AND MAINTENANCE MANUAL

Operation and Maintenance is not required for this redevelopment.



APPENDIX A



APPENDIX B

From: King Chin [mailto:kchin@geoengineers.com]

Sent: Thursday, March 09, 2017 2:30 PM
To: Scott Sherrow < scotts@paceengrs.com >
Subject: RE: Washington Cathedral Site

Hi, Scott. Based on the geologic map, the Washington Cathedral site is underlain by glacial till and transitional bed, both material is practically impermeable and is unfavorable for an infiltration facilities.

Thanks,

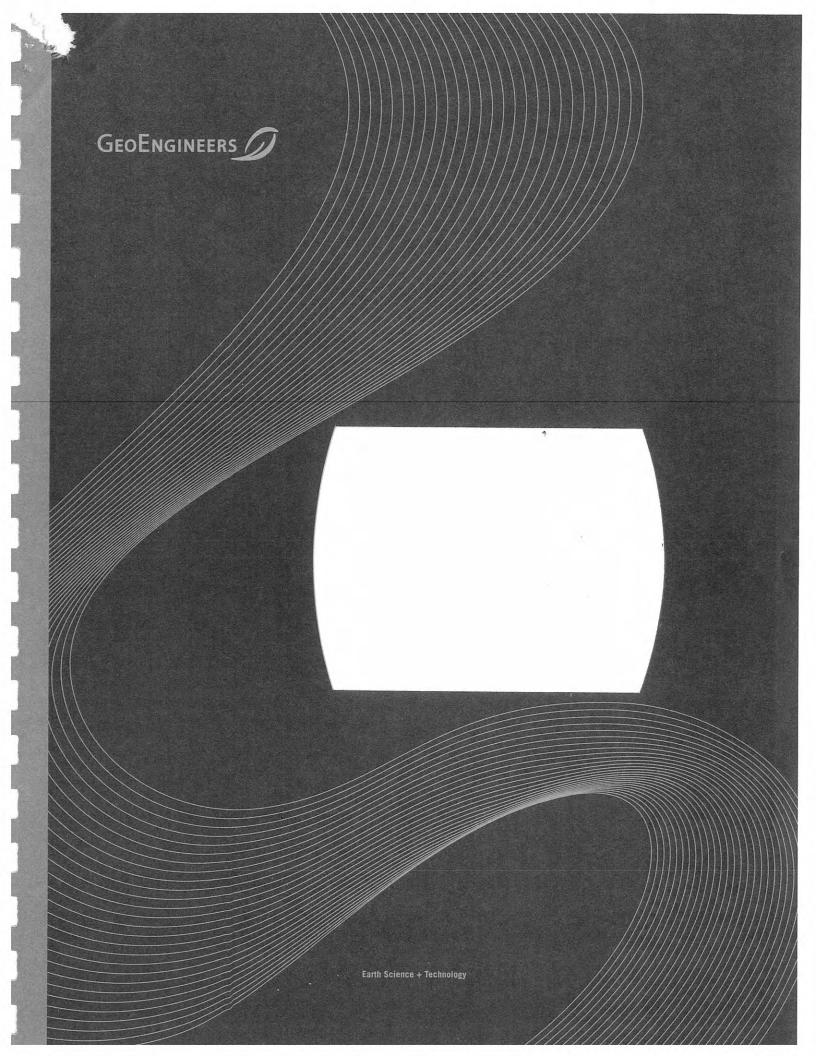
king

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Report
Geotechnical Engineering Services
Proposed Community Center and
Gymnasium Complex Building
Redmond, Washington

July 26, 2004

For Washington Cathedral



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EXECUTIVE SUMMARY

This report contains the results of our geotechnical engineering services for use in the design of the proposed community center at Washington Cathedral in Redmond Washington. The project consists of development of a gymnasium complex, a new sanctuary, a retirement and transitional housing building and two buildings that will house preschool, elementary school and Sunday school classrooms.

SUBSURFACE CONDITIONS

The subsurface conditions at the site were evaluated by excavating nineteen test pits throughout the site. Based on our explorations, the near-surface soils generally consist of fill overlying glacial till that is underlain by transitional bed deposits. Fill consists of medium stiff silt and clay. The glacial till consists of dense to very dense silty sand with gravel. The transitional bed deposits consist of very stiff to hard silt and clay. Slow to moderate groundwater seepage was observed in two of the test pits (TP-10 and TP-15).

SEISMIC DESIGN

In accordance with the 1997 Uniform Building Code (UBC), the site is classified as a Soil Type Sc. Likewise, in accordance with the 2003 International Building Code (IBC), the site is classified as Soil Profile Type C.

FOUNDATION DESIGN

The proposed gymnasium complex building and other planned buildings can be supported on conventional spread footings bearing on undisturbed very stiff to hard native transitional bed deposits or dense to very dense native glacial till or on structural fill placed over these native soils. Footings bearing on native transitional bed deposits may be designed using an allowable soil bearing value of 4,000 psf. Footings bearing on native glacial till may be designed using an allowable soil bearing value of 6,000 psf. Footings bearing on structural fill placed over undisturbed native soils may be designed using an allowable soil bearing value of 3,000 psf. The allowable bearing value may be increased by one-third for short duration loads such as wind or seismic events.

Lateral foundation loads may be resisted by passive resistance on the sides of the footings and by friction on the base of the footings. For footings supported and surrounded by either stiff or dense native soils or compacted structural fill, a coefficient of friction of 0.35 and a passive resistance of 350 pounds per cubic foot (pcf) may be used.

Footing drains should be incorporated in the design of buildings and retaining walls.

FLOOR SLABS

A subgrade modulus of 200 pounds per cubic inch (pci) may be used for design of the slabs-on-grade at the site. Concrete slabs-on-grade should be supported on a 6-inch-thick capillary break layer overlain by a vapor retarder. The basement floor slab should be provided with an under drain system.

BELOW-GRADE WALLS AND RETAINING WALLS

Below-grade walls and retaining walls should be provided with a free draining drainage layer and footing drain pipes. For below-grade walls constructed either neat against the dense native soils, or backfilled with compacted structural fill, we recommend the following equivalent fluid weights:

- allowable passive 350 pcf
- active 35 pcf
- at rest 55 pcf.

TEMPORARY CUT SLOPES AND SHORING

Excavations for the below grade level for the gymnasium complex building will require cuts up to about 35 feet deep along the south side of the building, and at the southeast and southwest corners of the building. Cuts on the order of 4 to 17 feet will be needed for the rest of the building. These cuts can be made as temporary open cut slopes depending on site constraints. For temporary unsupported cut slopes more than 4 feet high, the slope may be inclined no steeper than 1H:1V (horizontal:vertical) within the dense to very dense glacial till and 1.5H:1V within the very stiff to hard transitional bed deposits and the existing fill.

Excavations for the deeper portions of the below grade level for the gymnasium complex building can also be achieved using temporary shoring like soldier pile and timber lagging. It may be more cost effective and less intrusive to the site especially near the footprint of the proposed future housing located near the southeast corner of the gymnasium complex building.

REUSE OF ON-SITE MATERIALS

The existing fill and native transitional bed deposits are expected to be suitable for use as structural fill only during the summer months and in areas needing only 90 percent compaction (ASTM D 1557) by utilizing good construction practices. We do not recommend using the native transitional bed deposits during wet weather. The glacial till may be used for structural fill only during dry weather.

NEW HMA PAVEMENTS

New hot mix asphalt (HMA) pavement sections around the building should consist of at least 2 inches HMA over 4 inches of base course in car parking areas and 3 inches HMA over 6 inches of base course in areas exposed to truck traffic or heavy traffic volumes. A sand and gravel subbase should be placed below the base coarse material to enhance long term performance of the pavement and improve drainage beneath the pavement section.

REPORT GEOTECHNICAL ENGINEERING SERVICES PROPOSED COMMUNITY CENTER AND GYMNASIUM COMPLEX BUILDING REDMOND, WASHINGTON FOR WASHINGTON CATHEDRAL

INTRODUCTION

This report presents the results of our geotechnical engineering services for the proposed community center project, including the gymnasium complex building, at Washington Cathedral located in Redmond, Washington. The location of the site is shown on the Vicinity Map, Figure 1. We have also completed a critical areas site assessment for the project site, the results of which are presented in our, "Report, Preliminary Critical Areas Site Assessment, Washington Cathedral Site, Redmond, Washington," dated July 14, 2004. The general site configuration and location of the proposed gymnasium complex building are shown on the Site Plan, Figure 2.

PROJECT DESCRIPTION

Our understanding of the project is based on discussions with Troy Howe of Fourfold Architecture PLLC and review of maps and drawings that were provided by him, which include: King County Sensitive Area Map of the project site, the conceptual Community Center Site Plan, the conceptual Gymnasium Complex Plan and Building Sections, and the Topographic Survey of the site that was prepared by Duane Hartman & Associates, Inc. We understand that the project site consist of two parcels, located southeast of the intersection of Redmond Woodinville Road and Northeast 124th Street; a 5-acre parcel that occupies the northern 1/3 of the site and a 10-acre parcel that occupies the southern 2/3 of the site. The planned community center will be constructed within the 10-acre parcel. An access road will extend along the east side of the 5-acre parcel to Northeast 124th Street. The 10-acre parcel and the eastern portion of the 5-acre site are referred to herein as the "site."

We understand that the community center project will include development of a gymnasium complex, a new sanctuary, a retirement and transitional housing building and two buildings that will house preschool, elementary school and Sunday school classrooms. We also understand that the development of the community center will be completed in phases. The first phase of the project will include overall civil work for the site and construction of the gymnasium complex building. The gymnasium complex building will be located east of the existing sanctuary. According to the conceptual plan and building sections, the gymnasium complex, building will be a two story structure that houses an administration office, a gymnasium and a swimming pool. The administration office will be located at the second floor of the complex with the ground floor being the gymnasium and swimming pool. The swimming pool will be located in the southern portion of the complex. The gymnasium will be partially and completely below grade at the south and west side, and daylight to the north and east sides to the planned parking areas. The finish floor of the swimming pool and the gymnasium will be about Elevation 100 feet. This will require excavations ranging from 12 to 35 feet below the existing ground surface along the south and west sides of the building. Building details for the other planned buildings are not known at this time.

PURPOSE AND SCOPE

The purpose of our services is to evaluate soil and groundwater conditions as a basis for developing specific design criteria for the geotechnical aspects of the proposed gymnasium complex building and for developing general recommendations for the planned future housing and sanctuary buildings. Field explorations and laboratory testing were performed to identify and evaluate subsurface conditions at the site in order to develop engineering recommendations for use in design of the project.

Our services were performed in general accordance with the scope presented in our proposal dated June 18, 2004. Written authorization to proceed with our initial services was provided by Washington Cathedral on June 28, 2004.

FIELD EXPLORATION AND LABORATORY TESTING FIELD EXPLORATION

Subsurface conditions were evaluated through a field exploration program that consisted of excavating and sampling nineteen test pits throughout the site using a rubber-tired backhoe. The locations of the test pits are shown on the Site Plan, Figure 2.

The test pits, designated TP-1 through TP-19, were advanced to depths ranging from 5 to 11 feet below the ground surface. Locations of the test pits were determined in the field by measuring to existing site features such as the access road, the existing building and pavement areas. Elevations were estimated from Topographic Survey of the site that was prepared by Duane Hartman & Associates dated September 10, 2003 and used as a base for our Site Plan, Figure 2. The respective ground surface elevations are shown on the test pit logs in Appendix A. Appendix A includes logs of the test pits (Figures A-1 through A-20) and details of the subsurface explorations performed.

LABORATORY TESTING

Soil samples obtained from the explorations were transported to our laboratory and examined to confirm or modify field classifications, as well as to evaluate engineering properties of the soil. Representative samples were selected for laboratory testing consisting of moisture content determinations. The tests were performed in general accordance with test methods of the American Society for Testing and Materials (ASTM) or other applicable procedures. Appendix A includes a brief discussion of the laboratory tests and test results.

SITE DESCRIPTION

SURFACE CONDITIONS

The site generally slopes down from about Elevation 175 feet in the south to about Elevation 50 feet along Northeast 124th Street. The south east portion of the site is currently occupied by three church office buildings. The existing sanctuary is located in the northwest portion of the site. The ground surface within the southeast portion of the site near the office buildings slopes gently down to the east and north for a distance of about 100 feet and then slopes steeply to the northwest to the existing parking lot and to the west to the Redmond-Woodinville Road. Based on our discussion with Mike Roth (the backhoe operator) and review of the site plan dated April 8, 1993 prepared by Wolter Design Group Architects/Planner, the steep slopes were partially created as a result of fill placement during the previous

development in 1993. An access road connecting the existing sanctuary and the office buildings is located along the north and east sides of the site. Part of the access road is paved with asphalt concrete and part of it is gravel surfaced.

SITE GEOLOGY

Geologic information for the project area ("Geologic Map of the Kirkland Quadrangle, Washington", by J.P. Minard, 1983) indicates that native surficial soils at the site are composed of glacial till, which are underlain by transitional bed deposits. Glacial till commonly consists of a very compact, poorly sorted, non-stratified mixture of clay, silt, sand, gravel and cobbles. Glacial till commonly appears gray on a fresh surface, while weathered glacial till may be brown to yellow in color. Till may include large boulders. Transitional bed deposits generally consists of massive to bedded medium gray to dark gray clay, silt and fine to very fine sand.

SUBSURFACE SOIL CONDITIONS Soil Conditions

Based on the soil conditions encountered in the test pits, the project site is generally underlain by fill overlying dense to very dense glacial till and very stiff to hard transitional bed deposits at depth. The soil units encountered in the test pits are described below.

Fill. Fill was observed in all test pits except test pits TP-1, TP-2, TP-9, TP-11, TP-12 and TP-17. The fill generally consisted of gray silt and clay with sand and gravel and varied from 1.5 to 8 feet thick. Fill was most likely placed during the grading activities for previous development at the site. The thickness of the fill encountered in each test pit is included in Figure 2.

Glacial Till. Dense to very dense glacial till was observed beneath the fill in test pits TP-1 through TP-3, TP-5 through TP-10, TP-13, and TP-15 through TP-17. The till generally consists of grayish brown silty fine to medium sand with varying amounts of gravel. Although not observed, cobbles and boulders are commonly encountered within the till deposits.

Transitional Bed Deposits. Transitional bed deposits consisting of very stiff to hard silt and clay were observed below the till in test pits TP-1, TP-3, and TP-17 and were observed below the fill in test pits TP-4, TP-14, TP-18 and TP-19. Transitional bed deposits were observed to the full depths explored in test pits TP-11 and TP-12. Portions of the transitional bed deposits were observed to be fractured and slickensided.

Groundwater Conditions

Slow to moderate groundwater seepage was observed in test pits TP-10 and TP-15 near the contact of fill and native soils. There was no groundwater seepage observed in the other test pits nor in the native soils. We anticipate that zones of perched groundwater will exist in fill layers over the less permeable native till and transitional bed deposits in response to seasonal changes in precipitation.

CONCLUSIONS AND RECOMMENDATIONS

EARTHWORK General

Based on the subsurface soil conditions encountered in the test pits, we expect that the soils at the site may be excavated using conventional heavy duty construction equipment. The materials we encountered generally include dense to very dense silty sand, and very stiff to hard silt and clay. The native glacial deposits may require a large excavator to accomplish the excavations. Glacial deposits in the area commonly contain cobbles and boulders that may be encountered during excavation. Accordingly, the contractor should be prepared to deal with boulders, if encountered.

The glacial soils contain significant fines (material passing the U.S. standard No. 200 sieve) and will be highly moisture-sensitive and susceptible to disturbance, especially when wet. Ideally, earthwork should be undertaken during extended periods of dry weather when the surficial soils will be less susceptible to disturbance and provide better support for construction equipment. Dry weather construction will help reduce earthwork costs and increase the potential for using the native soils as fill.

Trafficability on the site is not expected to be difficult during dry weather conditions. However, the native soils will be susceptible to disturbance from construction equipment during wet weather conditions and pumping and rutting of the exposed soils under equipment loads may occur.

Clearing and Site Preparation

Areas to be developed or graded should be cleared of surface and subsurface deleterious matter including any debris, shrubs, trees and associated stumps and roots. Graded areas should also be stripped of organic soils.

The organic soils can be stockpiled and used later for landscaping purposes or may be spread over disturbed areas following completion of grading. If spread out, the organic strippings should be in a layer less that 1 foot thick, should not be placed on slopes greater than 3H:1V and should be track-rolled to a uniformly compacted condition. Materials that cannot be used for landscaping or protection of disturbed areas should be removed from the project site.

Subgrade Preparation

Prior to placing new fills, pavement base course materials, or gravel to support on-grade floor slabs, subgrade areas should be proofrolled to locate any soft or pumping soils. Prior to proofrolling, all unsuitable soils should be removed from below building areas. Proofrolling can be completed using a piece of heavy tire-mounted equipment such as a loaded dump truck. During wet weather, the exposed subgrade areas should be probed to determine the extent of soft soils. If soft or pumping soils are observed they should be removed and replaced with structural fill.

If deep pockets of soft or pumping soils are encountered outside the building area, it may be possible to limit the depth of overexcavation by placing a non-woven geotextile fabric such as Mirafi 500X (or similar material) on the overexcavated subgrade prior to placing structural fill. The geotextile will provide additional support by bridging over the soft material and will help reduce fines contamination into the structural fill.

After completing the proofrolling, the subgrade areas should be recompacted to a firm and unyielding condition, if possible. The degree of compaction that can be achieved will depend on when the construction is performed. If the work is performed during dry weather conditions, we recommend that all subgrade areas be recompacted to at least 95 percent of the maximum dry density (MDD) in accordance with the American Society for Testing and Materials (ASTM) D 1557 test procedure (modified Proctor). If the work is performed during wet weather conditions, it may not be possible to recompact the subgrade to 95 percent of the MDD. In this case, we recommend that the subgrade be compacted to the extent possible without causing undue weaving or pumping of the subgrade soils.

Subgrade disturbance or deterioration could occur if the subgrade is wet and can not be dried. If the subgrade deteriorates during proofrolling or compaction, it may become necessary to modify the proofrolling or compaction criteria or methods.

Working Pad

If construction of the building occurs during the wet weather months, generally October through May, routing of equipment on the subgrade soils will be difficult and the subgrade will likely become disturbed and softened. In addition, a significant amount of mud can be produced. Therefore, to protect the subgrade soils and to provide an adequate working surface for the contractor's equipment and labor, we recommend that a working pad layer be placed over the exposed subgrade soils. The working pad layer should be about 12 inches thick and should consist of 1½-inch minus clean crushed gravel with negligible sand or silt. The working pad layer can be placed in one lift and should be compacted to at least 95 percent of the MDD (per ASTM D 1557). If a working pad layer is used, and if relatively clean immediately prior to construction of the slab-on-grade, it may be used to replace the 6-inch capillary break layer. However, if the working pad becomes contaminated with soil, the contaminated areas should be overexcavated to a depth of 6 inches for placement of the capillary break gravel prior to constructing slabs-on-grade.

Structural Fill

All soil that will be used as fill, whether existing fill, on-site glacial till soil and transitional bed deposits, or imported soil, that will support floor slabs, pavement areas or foundations, or be placed against retaining walls or in utility trenches should generally meet the criteria for structural fill presented below. The suitability of soil for use as structural fill depends on its gradation and moisture content.

Materials. Materials used to construct building pads, embankments and surface parking areas are classified as structural fill for the purpose of this report. Structural fill material quality varies depending upon its use as described below:

Structural fill placed to construct embankment and parking areas, to backfill utility trenches and to support building and floor slabs may consist of on-site glacial till or transitional bed deposits provided that the soils are conditioned for the required compaction. On-site transitional bed deposits will likely be suitable for use as structural fill during dry weather conditions in areas needing 90 percent compaction. The on-site glacial till may be suitable for use as structural fill during dry

weather where compaction to 95 percent is needed. If structural fill is placed during wet weather, the structural fill should consist of imported gravel borrow as described in Section 9-03.14(1) of the 2004 Washington State Department of Transportation (WSDOT) Standard Specifications, with the additional restriction that the fines content be limited to no more than 5 percent. It is possible to use on-site till during wet weather for areas requiring only 90 percent compaction provided the earthwork contractor implements good wet weather techniques.

- 2. Structural fill placed immediately outside below-grade walls (drainage zone) should consist of washed 3/8 inch to No. 8 pea gravel or conform to Section 9-03.12(4) of the 2004 WSDOT Standard Specifications, as shown on Figure 3.
- 3. Structural fill placed as crushed surfacing base course below pavements should conform to Section 9-03.9(3) of the 2004 WSDOT Standard Specifications.
- 4. Structural fill placed as capillary break below slabs should consist of 1½-inch minus clean crushed gravel with negligible sand or silt in conformance with Section 9-03.1(4)C, grading No. 57 of the 2004 WSDOT Standard Specifications.

Reuse of On-site Soils. The existing fill, and native till and transitional bed deposits contain a high percentage of fines and will be sensitive to changes in moisture content and difficult to handle and compact during wet weather.

The glacial till deposits are expected to be suitable for structural fill in areas requiring compaction to at least 95 percent of MDD (per ASTM D 1557), provided the work is accomplished during the normally dry season (June through September) and that the soil can be properly moisture conditioned. It may be necessary to import sand and gravel with a low fines content to achieve adequate compaction for support of pavement areas, floor slabs and structures for wet weather construction. Imported structural fill consisting of sand and gravel (WSDOT gravel borrow) should be planned under all building floor slabs and foundation elements and if construction occurs during wet weather.

The use of existing on-site till as structural fill during wet weather should be planned only for areas requiring compaction to 90 percent of MDD, as long as the soils are properly protected from wet weather and not placed during periods of precipitation. The contractor should plan to cover and maintain all fill stockpiles with plastic sheeting if it will be used as structural fill. The reuse of on-site soils is highly dependent on the skill of the contractor and schedule, and we will work with the design team and contractor to maximize the reuse of on-site till soils during the wet and dry seasons.

The existing fill and transitional hed deposits are expected to be suitable for structural fill only during the normally dry season and then only in areas requiring compaction to 90 percent of MDD.

Fill Placement and Compaction Criteria. Structural fill should be mechanically compacted to a firm, non-yielding condition. Structural fill should be placed in loose lifts not exceeding 8 to 10 inches in thickness. The actual thickness will be dependent on the structural fill material used and the type and size of compaction equipment. Each lift should be conditioned to the proper moisture content and compacted to the specified density before placing subsequent lifts. Structural fill should be compacted to the following criteria:

- 1. Structural fill placed behind below-grade walls should be compacted to between 90 to 92 percent of the MDD estimated in accordance with ASTM D 1557. Care should be taken when compacting fill near the face of below-grade walls to avoid over-compaction and hence overstressing the walls. Hand operated compactors should be used within 5 feet behind the wall. Wall backfill placed within the building footprint and under the second floor level should be compacted to between 90 to 92 percent of the MDD within 5 feet of the walls and to at least 95 percent of the MDD beyond 5 feet of the walls. The upper 2 feet of fill below floor slab subgrade should also be compacted to at least 95 percent of the MDD. The contractor should keep all heavy construction equipment away from the top of retaining walls a distance equal to ½ the height of the wall, or at least 5 feet, which ever is greater.
- 2. Structural fill in new pavement and hardscape areas, including utility trench backfill, should be compacted to at least 90 percent of the MDD estimated in accordance with ASTM D 1557, except that the upper 2 feet of fill below final subgrade should be compacted to at least 95 percent of the MDD, see Figure 4.
- 3. Structural fill placed below floor slabs and foundations should be compacted to 95 percent of the MDD estimated in accordance with ASTM D 1557.
- 4. Structural fill placed as crushed rock base course below pavements should be compacted to 95 percent of the MDD estimated in accordance with ASTM D 1557.
- 5. Non-structural fill, such as fill placed in landscape areas, should be compacted to at least 85 percent of the MDD estimated in accordance with ASTM D 1557. In areas intended for future development, a higher degree of compaction should be considered to reduce the settlement potential of the fill soils.

Weather Considerations. Disturbance of near surface soils should be expected if earthwork is completed during periods of wet weather. During dry weather the soils will (1) be less susceptible to disturbance, (2) provide better support for construction equipment, and (3) be more likely to meet the required compaction criteria.

The wet weather season generally begins in October and continues through May in western Washington; however, periods of wet weather may occur during any month of the year. For earthwork activities during wet weather, we recommend that the following steps be taken:

- The ground surface in and around the work area should be sloped so that surface water is directed away from the work area. The ground surface should be graded so that areas of ponded water do not develop. Measures should be taken by the contractor to prevent surface water from collecting in excavations and trenches. Measures should be implemented to remove surface water from the work area.
- Earthwork activities should not take place during periods of moderate to heavy precipitation.
- Slopes with exposed soils should be covered with plastic sheeting.
- The contractor should take necessary measures to prevent on-site soils and soils to be used as fill from becoming wet or unstable. These measures may include the use of plastic sheeting, sumps with pumps, and grading. The site soils should not be left uncompacted and exposed to moisture. Sealing

the surficial soils by rolling with a smooth-drum roller prior to periods of precipitation will help reduce the extent that these soils become wet or unstable.

- The contractor should cover all soil stockpiles that will be used as structural fill with plastic sheeting.
- Construction traffic should be restricted to specific areas of the site, preferably areas that are surfaced
 with the existing asphalt or working pad materials not susceptible to wet weather disturbance.
- Construction activities should be scheduled so that the length of time that soils are left exposed to
 moisture is reduced to the extent practical.

Routing of equipment on the subgrade soils during the wet weather months will be difficult and the subgrade will likely become highly disturbed and rutted. In addition, a significant amount of mud can be produced by routing equipment directly on the glacial soils in wet weather. Therefore, to protect the subgrade soils and to provide an adequate wet weather working surface for the contractor's equipment and labor, we recommend that the contractor protect exposed subgrade soils with sand and gravel, crushed gravel, or ATB.

Permanent Cut and Fill Slopes

We recommend that permanent cut or fill slopes be constructed at inclinations of 2H:1V or flatter, and be blended into existing slopes with smooth transitions. To achieve uniform compaction, we recommend that fill slopes be overbuilt slightly and subsequently cut back to expose well-compacted fill.

To reduce erosion, newly constructed slopes should be planted or hydroseeded shortly after completion of grading. Until the vegetation is established, some sloughing and ravelling of the slopes should be expected. This may necessitate localized repairs and reseeding. Temporary covering, such as clear heavy plastic sheeting, jute fabric, or erosion control blankets (such as American Excelsior Curlex 1 or North American Green S150) could be used to protect the slopes during periods of rainfall.

Utility Trenches

Trench excavation, pipe bedding, and trench backfilling should be completed using the general procedures described in the 2004 WSDOT Standard Specifications or other suitable procedures specified by the project civil engineer. The native glacial deposits and fill soils encountered at the site are generally of low corrosivity based on our experience in the Puget Sound area.

Utility trench backfill should consist of structural fill and should be placed in lifts of 8 inches or less (loose thickness) such that adequate compaction can be achieved throughout the lift. Sand backfill, containing less than 5 percent fines, may be compacted in loose lifts not exceeding 12 inches when placed below five feet of the finished ground surface. Each lift must be compacted prior to placing the subsequent lift. Prior to compaction, the backfill should be moisture conditioned to within 3 percent of the optimum moisture content, if necessary. The backfill should be compacted in accordance with the criteria discussed above. Figure 4 illustrates recommended trench compaction criteria under pavement and non-structural areas.

Sedimentation and Erosion Control

In our opinion, the erosion potential of the on-site soils is moderate to high because of the presence of the steep slopes towards the north and west site boundaries. Construction activities including stripping and grading will expose soils to the erosional effects of wind and water. The amount and potential impacts of erosion are partly related to the time of year that construction actually occurs. Wet weather construction will increase the amount and extent of erosion and potential sedimentation.

Erosion and sedimentation control measures may be implemented by using a combination of interceptor swales, straw bale barriers, silt fences and straw mulch for temporary erosion protection of exposed soils. All disturbed areas should be finish graded and seeded as soon as practicable to reduce the risk of erosion. Erosion and sedimentation control measures should be installed and maintained in accordance with the requirements of the City of Redmond.

EXCAVATIONS General

Excavations for the gymnasium complex building will require cuts up to about 35 feet deep along the south, southeast and southwest sides of the building. Cuts on the order of 4 to 17 feet will be needed for the rest of the building. These cuts can likely be made as temporary open cut slopes provided the cut slopes do not affect adjacent structures or require significant backfill thicknesses where future buildings are planned (i.e. housing to be located southeast of the gymnasium complex). Excavations are also required for underground utilities. The stability of open cut slopes is a function of soil type, groundwater seepage, slope inclination, slope height and nearby surface loads. The use of inadequately designed open cuts could impact the stability of adjacent work areas, existing utilities, and endanger personnel.

The contractor performing the work has the primary responsibility for protection of workmen and adjacent improvements. In our opinion, the contractor will be in the best position to observe subsurface conditions continuously throughout the construction process and to respond to variable soil and groundwater conditions. Acceptable slope inclinations for utilities and ancillary excavations should be determined during construction. Because of the diversity of construction techniques and available shoring systems, the design of temporary shoring is most appropriately left up to the contractor proposing to complete the construction. Temporary cut slopes and shoring must comply with the provisions of Title 296 WAC, Part N, "Excavation, Trenching and Shoring."

The excavations for below grade level of the gymnasium complex building will be completed primarily in fill, dense to very dense glacial till, and stiff to hard transitional bed silt/clay. The following sections summarize the general excavation recommendations. If conditions allow, the entire excavation for the proposed building may be accomplished using temporary cut slopes. However, depending on the final building layout and site constraints, including the planned location of future buildings, temporary shoring may also be needed. Temporary shoring appropriate for the soil conditions at the site may include soldier pile and timber lagging walls. Temporary retaining walls must account for groundwater seepage in their design and during construction. In the following sections we provide a brief description of soldier pile and timber lagging walls.

Temporary Cut Slopes

For planning purposes, temporary cut slopes made in the very dense glacial till may be inclined at 1H:1V. Dense till is expected in some of the excavations for the gymnasium complex building. Excavations made in interbedded transitional bed deposits should be made no steeper than 1.5H:1V due the presence of water bearing sand layers and fissured nature of these soils. Transitional bed deposits will be encountered along the lower portions of the cut slope at the gymnasium complex building. Looser fill soils overlying the native till or transitional bed deposits should be excavated no steeper than 1.5H:1V.

The above guidelines assume that surface loads such as traffic, construction equipment, stockpiles or building supplies will be kept away from the top of the cut slopes a sufficient distance so that the stability of the excavation is not affected. We recommend that this distance be at least 5 feet from the top of the cut for temporary cuts made at 1H:1V or flatter. Temporary cut slopes should be planned such that they do not encroach on a 1H:1V influence line projected down from the edges of nearby or planned foundation elements.

Water that enters the excavation must be collected and routed away from prepared subgrade areas. We expect that this may be accomplished by installing a system of shallow drainage ditches and sumps along the toe of the cut slopes. Some sloughing and raveling of the cut slopes should be expected. Temporary covering, such as heavy plastic sheeting with appropriate ballast, should be used to protect these slopes during periods of wet weather. Surface water runoff from above cut slopes should be prevented from flowing over the slope face by using berms, drainage ditches, swales or other appropriate methods.

If temporary cut slopes experience excessive sloughing or raveling during construction, it may become necessary to modify the cut slopes to maintain safe working conditions. Slopes experiencing problems can be flattened, regraded to add intermediate slope benches, or additional dewatering can be provided if the poor slope performance is related to groundwater seepage.

Soldier Pile and Timber Lagging Walls

Soldier Piles. It may be desirable to use temporary shoring to limit the extent of excavations. We recommend that temporary shoring consist of cantilevered soldier pile and timber lagging walls and that walls for the gymnasium complex building excavations be designed using the earth pressure diagram presented in Figures 5.

We recommend that the embedded portion of the soldier piles extend a minimum distance of 10 feet below the base of the excavations or bottom of adjacent subdrain trenches to resist "kick-out." If tied-back walls are required then the axial capacity of the soldier pile must resist downward components of anchor loads and other vertical loads. We recommend using an allowable end-bearing value of 15 kips per square foot (ksf) for piles supported on the dense to very dense till and transitional bed deposits. The allowable end bearing value should be applied to the base area of the drilled hole into which the soldier pile is concreted. This value includes a factor of safety of about 3. The allowable capacity may be increased by one-third for short-term loads, such as seismic forces. A side resistance value of 1,000 psf can be used for soldier piles below the base of excavations in till or transitional bed deposits. We should be contacted for additional recommendations if tieback anchors are required.

Seismic pressure does not need to be included in the design of temporary walls; however, it should be included if the wall will be used as part of the permanent building wall or as a permanent shoring system.

Casing and/or slurry may be required to install the soldier piles due to the presence of perched ground water conditions, deeper ground water (such as the interbedded transitional bed deposits). The need for and the use of casing for installation of the soldier piles should be determined by the shoring contractor based on the subsurface conditions presented in this report. The contractor should make sure that heave or disturbance of the native soil at the bottom of the soldier pile holes does not occur, especially in transitional bed deposits at the site. Observations made at the site by the General Contractor for the various soil units encountered will also be important to the shoring contractor. Also, boulders may be present in the glacial soils. The contractor should be prepared to address the presence of boulders during construction.

Lagging. We recommend that the timber lagging be designed for uniform pressures equal to one-half the active lateral pressures as shown on Figure 5. This pressure reduction is based on a maximum center-to-center pile spacing of 8 feet. If a wider spacing is desired, we should be consulted for revised lagging pressures.

Permanent lagging should be installed between the soldier piles to retain the soils where the wall will be used as a permanent retaining structure. Permanent lagging may consist of timber or concrete. If timber is used, it must be adequately treated for protection against water and biodegradation.

Drainage. Drainage may be accomplished by spacing the timber lagging with a vertical gap of approximately 3/8 inch. If the soldier pile wall will provide the form for the new building wall, then wick drains or strips of drainage material, such as Miradrain, should also be installed in front of the lagging. The strip drains should be at least 18-inches wide and extend the entire height of the wall. We recommend that strip drains be connected to a perforated drain installed along the base of the wall and routed to a suitable discharge point as described below in "Drainage Considerations."

The space behind the lagging must be filled with free draining material as soon as practical. The free draining material will help reduce the risk of voids behind the wall and provide additional drainage of potential groundwater seepage. The free draining material should be poorly graded sand with no particle larger than 1/4 inch nor smaller than the U.S. Standard No. 40 sieve.

EARTHQUAKE ENGINEERING General

GeoEngineers has evaluated the site for seismic hazards including liquefaction, lateral spreading, fault rupture and earthquake induced landsliding. Our evaluation indicates that the site does not have liquefiable soils present and therefore also has no risk of liquefaction induced lateral spreading. In addition the site has a low risk of fault rupture and earthquake induced landsliding.

1997 UBC Seismic Design Information

The Puget Sound region is designated as a Seismic Zone 3 in the 1997 edition of the Uniform Building Code (UBC). For Zone 3 locations, including the site, we recommend the 1997 UBC Soil Profile Type, and Seismic Coefficients presented in Table 1.

Table 1. UBC Seismic Parameters

1997 UBC Parameter	Recommended Valu		
Soil Profile Type	S _c		
Seismic Coefficient, Ca	0.33		
Seismic Coefficient, C _v	0.45		

2003 IBC Seismic Design Information

For the site, we recommend the 2003 International Building Code (IBC) parameters for Soil Profile Type, short period spectral response acceleration (SS), 1-second period spectral response acceleration (S1), and Seismic Coefficients FA and FV presented in Table 2.

Table 2. IBC Seismic Parameters

2000 IBC Parameter	Recommended Value
Soil Profile Type	С
Short Period Spectral Response Acceleration, S _S (percent g)	127.4
1-Second Period Spectral Response Acceleration, S ₁ (percent g)	58.2
Seismic Coefficient, F _A	1.0
Seismic Coefficient, F _V	1.37

FOUNDATIONS General

We recommend that the proposed gymnasium complex building and the other planned buildings at the site be supported on shallow spread footings founded on undisturbed dense to very dense glacial till or very stiff to hard transitional bed deposits encountered in our test pits or on properly compacted structural fill extending down to the dense to very dense or very stiff to hard glacial deposits. Foundations should not be supported on the existing fill soils. If structural fill is used to support foundations then the zone of structural fill should extend beyond the faces of the footing a distance at least equal to the thickness of the structural fill.

Foundation Design

For shallow foundation support, we recommend widths of at least 24 and 36 inches, respectively, for continuous wall and isolated column footings supporting the proposed building. Provided that footings are supported as recommended above, an allowable bearing value of 6,000 psf may be used for the footings supported on the dense to very dense glacial till deposits. Footings supported on very stiff to hard transitional bed deposits may be designed for an allowable bearing capacity of 4,000 psf. Footings supported on structural fill compacted to at least 95 percent MDD may be designed for an allowable

bearing capacity of 3,000 psf. These allowable bearing pressures apply to the total dead and long-term live loads and may be increased up to one-third for short-term live loads such as wind or seismic forces.

Exterior footings for the building should be founded at least 18 inches below lowest adjacent finished grade. Interior footings should be founded at least 12 inches below bottom of slab or adjacent finished grade.

Foundation Settlement

We estimate that the total postconstruction settlement of footings founded on the dense to very dense till deposits or very stiff to hard transitional bed deposits or structural fill extending to the very dense glacial deposits, as recommended above, should be less than 3/4 inch. Differential settlement between comparably loaded column footings or along a 25-foot section of continuous wall footing should be less than ½ inch. We expect most of the footing settlements will occur as loads are applied. Loose or disturbed soils not removed from footing excavations prior to placing concrete will result in additional settlement.

Construction Considerations

Immediately prior to placing concrete, all debris and loose soils that accumulated in the footing excavations during forming and steel placement must be removed. Debris or loose soils not removed from the footing excavations will result in increased settlement.

If wet weather construction is planned, we recommend that all footing subgrades be protected using a lean concrete mud mat. The mud mat should be placed the same day that the footing subgrade is excavated and approved for foundation support.

Lateral Resistance

Lateral loads can be resisted by passive resistance on the sides of the footings and by friction on the base of the footings. Passive resistance should be evaluated using an equivalent fluid density of 350 pcf where footings are poured neat against native soil or are surrounded by structural fill compacted to at least 95 percent of MDD, as recommended. Resistance to passive pressure should be calculated from the bottom of adjacent floor slabs and paving or below a depth of 1 foot where the adjacent area is unpaved, as appropriate. Frictional resistance can be evaluated using 0.35 for the coefficient of base friction against footings. The above values incorporate a factor of safety of about 1.5.

If soils adjacent to footings are disturbed during construction, the disturbed soils must be recompacted, otherwise the lateral passive resistance value must be reduced.

Footing Drains

We recommend that perimeter footing drains be installed around the gymnasium complex building and all future buildings. The perimeter drains should be installed at the base of the exterior footings. The perimeter drains should be provided with cleanouts and should consist of at least 4-inch-diameter perforated pipe placed on a 4-inch bed of, and surrounded by, 6 inches of drainage material enclosed in a non-woven geotextile fabric such as Mirafi 140N (or approved equivalent) to prevent fine soil from migrating into the drain material, as shown on Figure 3. The footing drain pipe should be installed at

least 18 inches below the top of the adjacent floor slab. The drainage material should consist of washed 3/8-inch to No. 8 pea gravel or "Gravel Backfill for Drains" per Section 9-03.12(4) of the WSDOT Standard Specifications. We recommend that the drainpipe consist of either heavy-wall solid pipe (SDR-35 PVC, or equal) or rigid corrugated smooth interior polyethylene pipe (ADS N-12, or equal). We also recommend against using flexible tubing for footing drainpipes. The perimeter drains should be sloped to drain by gravity, if practicable, to a suitable discharge point, preferably a storm drain. We recommend that the cleanouts be covered, and be placed in flush mounted utility boxes. Water collected in roof downspout lines <u>must not</u> be routed to the footing drain lines.

BELOW-GRADE WALLS AND RETAINING WALLS Design Parameters

The following recommendations should be used for the design of below-grade walls that are intended to act as retaining walls and for other retaining structures that are used to achieve grade changes.

Lateral earth pressures for design of below-grade walls and retaining structures should be evaluated using an equivalent fluid density of 35 pcf provided that the walls will not be restrained against rotation when backfill is placed. If the walls will be restrained from rotation, we recommend using an equivalent fluid density of 55 pcf. Walls are assumed to be restrained if top movement during backfilling is less than H/1000, where H is the wall height. These lateral soil pressures assume that the ground surface behind the wall is horizontal. For unrestrained walls with backfill sloping up at 2H:1V, the design lateral earth pressure should be increased to 55 pcf, while restrained walls with a 2H:1V sloping backfill should be designed using an equivalent fluid density of 75 pcf. These lateral soil pressures do not include the effects of surcharges such as floor loads, traffic loads or other surface loading. Surcharge effects should be included as appropriate. Below-grade walls for buildings should also include seismic earth pressures. Seismic earth pressures should be determined using a rectangular distribution of 8H in psf, where H is the wall height.

If vehicles can approach the tops of exterior walls to within 1/2 the height of the wall, a traffic surcharge should be added to the wall pressure. For car parking areas, the traffic surcharge can be approximated by the equivalent weight of an additional 1 foot of soil backfill (about 125 psf) behind the wall. For delivery truck parking areas and access driveway areas, the traffic surcharge can be approximated by the equivalent weight of an additional 2 feet (250 psf) of soil backfill behind the wall. Positive drainage should be provided behind below-grade walls and retaining structures as discussed below.

These recommendations are based on the assumption that any retaining walls at this project will be provided with adequate drainage. The values for soil bearing, frictional resistance and passive resistance presented above for foundation design are applicable to retaining wall design. Walls located in level ground areas should be founded at a depth of 18 inches below the adjacent grade.

Wall Drainage

To reduce the potential for hydrostatic water pressure buildup behind the retaining walls, we recommend that the walls be provided with adequate drainage, as shown in Figure 3. Wall drainage can

be achieved by using free draining wall drainage material with perforated pipes to discharge the collected water.

Wall drainage material may consist of washed 3/8-inch to No. 8 pea gravel or a mixture of about 30 to 40 percent clean medium to coarse sand and 60 to 70 percent fine gravel, with negligible fine material. Alternatively, the wall drainage material may consist of clean gravel (Gravel Backfill for Drains per Section 9-03.12(4) of the WSDOT Standard Specification) surrounded with a non-woven geotextile fabric such as Mirafi 140N (or approved equivalent). The zone of wall drainage material should be 2 feet wide and should extend from the base of the wall to within 2 feet of the ground surface. The wall drainage material should be covered with 2 feet of less permeable material, such as the on-site glacial till that is properly moisture conditioned and compacted.

A 4-inch-diameter perforated drain pipe should be installed within the free-draining material at the base of each wall. We recommend using either heavy-wall solid pipe (SDR-35 PVC) or rigid corrugated polyethylene pipe (ADS N-12, or equal). We recommend against using flexible tubing for the wall drain pipe. The footing drain recommended above can be incorporated into the bottom of the drainage zone and used for this purpose.

The pipes should be laid with minimum slopes of one-half percent and discharge into the stormwater collection system to convey the water off site. The pipe installations should include a cleanout riser with cover located at the upper end of each pipe run. The cleanouts could be placed in flush mounted access boxes. Collected downspout water should be routed to appropriate discharge points in separate pipe systems.

Other Considerations

Exterior retaining systems used to achieve grade transitions or for landscaping, can be constructed using traditional structural systems such as reinforced concrete, CMU blocks, or rockeries. Alternatively, retaining walls can consist of reinforced soil and block facing structures. In recent years, the latter structural system has proven to be an economically reasonable alternative to more traditional retaining wall systems. We can provide additional design recommendations for reinforced soil and block facing structures, if requested.

SLAB-ON-GRADE FLOOR Subgrade Preparation

We recommend that concrete slabs-on-grade be constructed on a gravel layer to provide uniform support and drainage, and to act as a capillary break. We expect that slab-on-grade floors can be supported on the native dense to very dense till and very stiff to hard transitional bed deposits encountered in our test pits or on properly compacted structural fill extending down to these materials. Prior to placing the gravel layer, the subgrade should be proofrolled as described previously in the Earthwork section of this report. If necessary, the subgrade should be recompacted to a firm and unyielding condition.

Design Parameters

The gravel layer below slabs-on-grade should consist of 6 inches of clean crushed gravel, with a maximum particle size of 1½ inches and negligible sand or silt. If the 12-inch-thick working pad layer is constructed and if areas contaminated with soil are removed, then the working pad layer can be used as the capillary break layer described above. For slabs designed as a beam on an elastic foundation, a modulus of subgrade reaction of 200 pci may be used for subgrade soils prepared as recommended above.

If water vapor migration through the slabs is objectionable, the gravel should be covered with a heavy plastic sheet, such as 10-mil plastic sheeting, to act as a vapor retarder. This will be desirable where the slabs will be surfaced with tile or will be carpeted. The contractor should be made responsible for maintaining the integrity of the vapor barrier during construction. A 2-inch-thick layer of sand should be placed over the vapor barrier to protect it during construction and to aid in uniform curing of the concrete. The structural engineer should be consulted if the contractor proposes deleting the 2-inch-thick layer of sand. It may also be prudent to apply a sealer to the slab to further retard the migration of moisture through the floor.

PAVEMENT RECOMMENDATIONS Subgrade Preparation

We recommend that the subgrade soils in new pavement areas be prepared and evaluated as described in the "Earthwork" section of this report. In cut areas in medium dense to dense native soils, we recommend that the upper 12 inches of the existing site soils be compacted to at least 95 percent of the MDD per ASTM D 1557 prior to placing pavement section materials. If the subgrade soils are loose or soft, it may be necessary to excavate the soils and replace them with structural fill, gravel borrow, or gravel base material. Pavement subgrade conditions should be observed and proof-rolled during construction to evaluate the presence of unsuitable subgrade soils and the need for over-excavation and placement of a geotextile fabric.

New Hot Mix Asphalt (HMA) Pavement

In light-duty pavement areas (e.g., automobile parking), we recommend a pavement section consisting of at least a 2-inch thickness of ½-inch HMA (PG 58-22) per Sections 5-04 and 9-03 of the WSDOT Standard specifications, over a 4-inch thickness of densely compacted crushed rock base course per Section 9-03.9(3) of the WSDOT Standard specifications. In heavy-duty pavement areas (e.g., truck traffic areas, materials delivery) around the building, we recommend a pavement section consisting of at least a 3-inch thickness of ½-inch HMA (PG 58-22) over a 6-inch thickness of densely compacted crushed rock base course. The base course should be compacted to at least 95 percent of the MDD per ASTM D 1557. We recommend that a proof-roll of the compacted base course be observed by a representative from our firm prior to paving. Soft or yielding areas observed during proof-rolling may require over-excavation and replacement with compacted structural fill. A layer of sand and gravel subbase may be placed below the base coarse material to enhance long term performance of the pavement and improve drainage beneath the pavement section.

The pavement sections recommended above are based on our experience. Thicker asphalt sections may be needed based on the actual traffic data and intended use.

DRAINAGE CONSIDERATIONS

We anticipate shallow groundwater seepage may enter deep excavations depending on the time of year construction takes place, especially in the winter months. However, we expect that this seepage water can be handled by digging shallow interceptor trenches in the excavations and pumping from sumps. The seepage water if not intercepted and removed from the excavations will make it difficult to place and compact structural fill and may destabilize cut slopes.

All paved and landscaped areas should be graded so that surface drainage is directed away from the building to appropriate catch basins.

Roof downspouts <u>should not</u> discharge into the perforated pipes intended for providing footing or wall drainage. Water collected in roof downspout lines <u>should</u> be routed to a suitable discharge point.

SOIL INFILTRATION CAPACITY

Soil infiltration capacity for the site soils were evaluated based on soil texture analysis from visual inspections. The soil infiltration rate was estimated in general accordance with the Ecology Stormwater Management Manual for Western Washington (Stormwater Management Manual, 2001). In our opinion, stormwater infiltration potential at this site is very limited because of surficial silty and silt/clay soil.

According to the Ecology Stormwater Management Manual, soil infiltration capacity can be estimated based on the grain-size/texture of the soil. Based on our visual inspection of the on site soil observed in the test pits, the on-site soils can generally be classified as silt or silt loam. According to the recommended infiltration rates based on USDA Soil Textural Classification in the Stormwater Management Manual, the silt or silt loam will have a long term design infiltration rate of much less than 0.13 inch/hr and are not recommended for infiltration facilities.

RECOMMENDED ADDITIONAL GEOTECHNICAL SERVICES

Throughout this report, recommendations are provided where we consider additional geotechnical services to be appropriate. These additional services are summarized below:

- GeoEngineers should be retained to review the project plans and specifications when complete to confirm that our design recommendations have been implemented as intended.
- During construction, GeoEngineers should observe temporary cut slopes, evaluate the suitability of the foundation subgrades, evaluate the suitability of floor slab and pavement subgrades, observe installation of subsurface drainage measures, observe and test structural backfill, and provide a summary letter of our construction observation services. The purposes of GeoEngineers construction phase services are to confirm that the subsurface conditions are consistent with those observed in the explorations and other reasons described in Appendix B, Report Limitations and Guidelines for Use.

LIMITATIONS

We have prepared this report for use by Washington Cathedral, Fourfold Architecture PLLC, and other members of the design team for use in design of this project.

Within the limitations of scope, schedule and budget, our services have been executed in accordance with generally accepted practices in the field of geotechnical engineering in this area at the time this report was prepared. No warranty or other conditions, express or implied, should be understood.

Any electronic form, facsimile or hard copy of the original document (email, text, table, and/or figure), if provided, and any attachments are only a copy of the original document. The original document is stored by GeoEngineers, Inc. and will serve as the official document of record.

Please refer to Appendix B titled Report Limitations and Guidelines for Use for additional information pertaining to use of this report.

We trust this report provides the information you require at this time. We appreciate the opportunity to be of service to you on this project and to provide services to Washington Cathedral. Please contact us should you have any questions concerning our findings or recommendations, or should you require additional information.

Yours very truly,

Geo Engineers, Inc

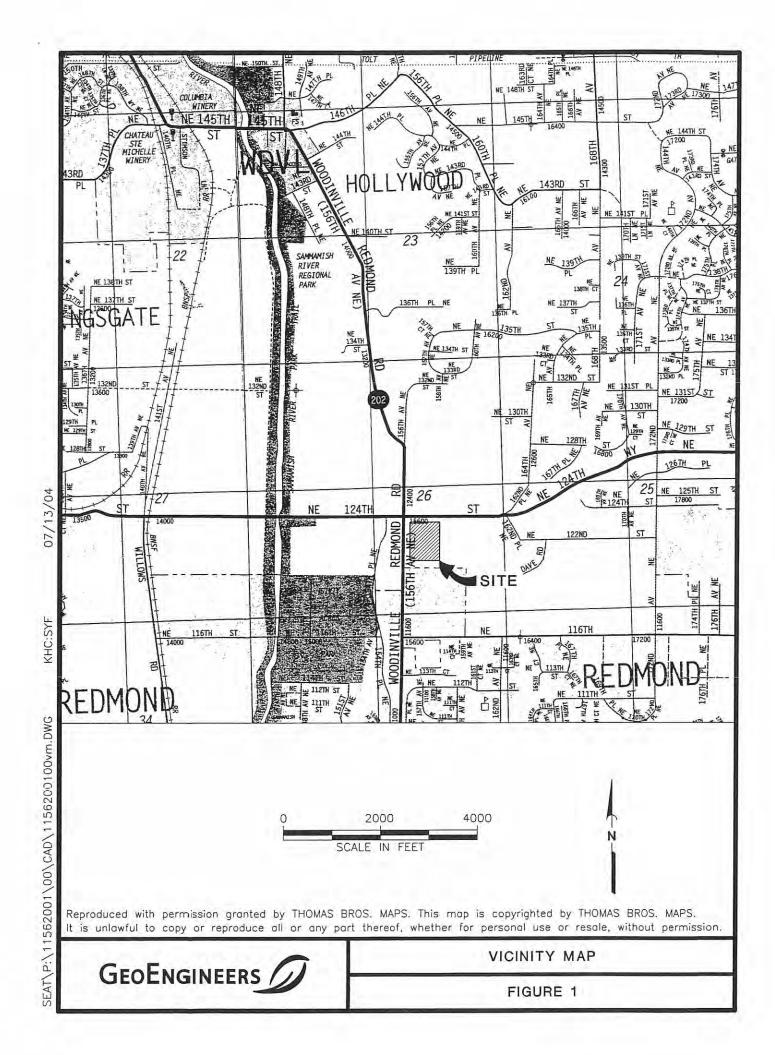
King H. Chin, PE Geotechnical Engineer/

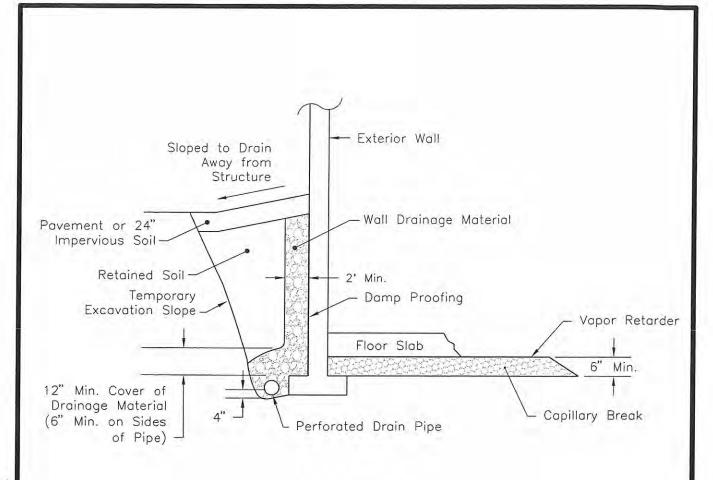
> Bo McFadden, PE, LEG Associate

KHC:JJM:ab

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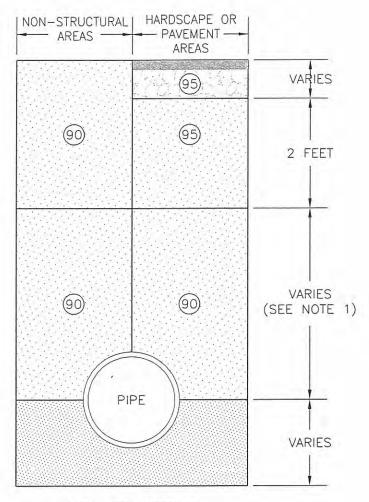


MATERIALS

- A. WALL DRAINAGE MATERIAL: May consist of washed 3/8" to No. 8 pea gravel. Alternatively the wall drainage material may consist of "Gravel Backfill for Drains" per WSDOT Standard Specification 9-03.12(4), surrounded with a non-woven geotextile such as Mirafi 140N (or approved equivalent).
- B. RETAINED SOIL: Should consist of structural fill, either on—site soil or imported. The backfill should be compacted in loose lifts not exceeding 6 inches. Wall backfill supporting building floor slabs should consist of imported sand and gravel per WSDOT Standard Specification 9—03.14 or City of Seattle Type 17 compacted to at least 95 percent ASTM D1557. Backfill not supporting building floor slabs, sidewalks, or pavement should be compacted to 90 to 92 percent of the maximum dry density, per ASTM D1557. Backfill supporting sidewalks or pavement areas should be compacted to at least 95 percent in the upper two feet. Only hand—operated equipment should be used for compaction within 5 feet of the walls and no heavy equipment should be allowed within 5 feet of the wall.
- C. CAPILLARY BREAK: Should consist of at least 6 inches of clean crushed gravel with a maximum size of 1-1/2 inches and negligible sand or fines.
- D. PERFORATED DRAIN PIPE: Should consist of a 4-inch diameter perforated heavy-wall solid pipe (SDR-35 PVC) or rigid corrugated polyethylene pipe (ADS N-12) or equivalent. Drain pipes should be placed with 0.5 percent minimum slopes and discharge to the storm water collection system.



WALL DRAINAGE AND BACKFILL



NOT TO SCALE

EXPLANATION:

95)

RECOMMENDED COMPACTION AS A PERCENTAGE OF MAXIMUM DRY DENSITY, BY TEST METHOD ASTM D1557 (MODIFIED PROCTOR)



CONCRETE OR ASPHALT PAVEMENT



BASE COURSE



TRENCH BACKFILL

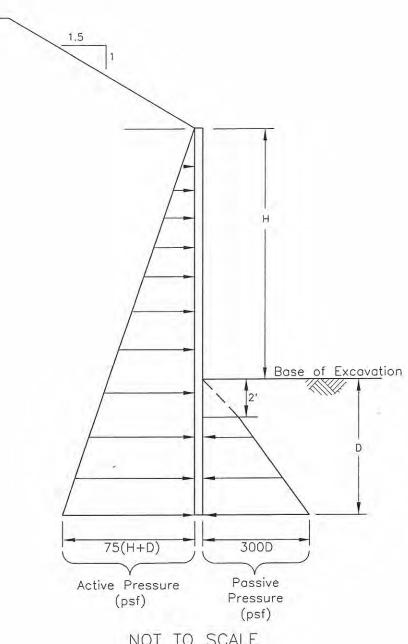


PIPE BEDDING

Note: 1. All backfill under building areas should be compacted to at least 95 percent per ASTM D1557.



COMPACTION CRITERIA FOR TRENCH BACKFILL



NOT TO SCALE

Notes: 1. Earth pressure coefficients are given in terms of equivalent fluid pressures (pounds per cubic foot).

2. Passive pressures are assumed to act over 2 times the soldier pile diameter or pile spacing, whichever is less, and include a factor of safety of 1.5.

3. Active pressures are assumed to act over pile spacing above base of excavation and over soldier pile diameter below base of excavation.

4. Seismic pressure does not need to be included in design of temporary walls.

5. Assumes no surcharge loads on slope.

6. Figure should only be used in conjunction with qualifications in report text.





APPENDIX A FIELD EXPLORATIONS AND LABORATORY TESTING

APPENDIX A

FIELD EXPLORATIONS AND LABORATORY TESTING FIELD EXPLORATIONS

Subsurface conditions at the site were explored on July 12, 2004 by excavating nineteen test pits (TP-1 through TP-21) at the approximate locations shown on the Site Plan, Figure 2. The test pits were excavated to depths ranging from about 5 to 11 feet. The tests pits were excavated using a Case 580 rubber-tired backhoe operated by Mike Roth. Test pit locations were determined in the field by measuring distances from existing site features.

The test pits were continuously monitored by a geotechnical engineer from our firm who examined and classified the soils encountered, obtained representative soil samples, and observed groundwater conditions. Our representative maintained a detailed log of each test pit. Disturbed samples of the representative soil types were obtained from the test pits.

Soils encountered in the test pits were classified in the field in general accordance with ASTM D 2488, the Standard Practice for Classification of Soils, Visual-Manual Procedure, which is summarized in Figure A-1. A key to the test pit log symbols is also presented in Figure A-1. Logs of the test pits are provided in Figures A-2 through A-20.

LABORATORY TESTING General

Soil samples obtained from the explorations were transported to our laboratory and examined to confirm or modify field classifications, as well as to evaluate engineering properties of the soil. Representative samples were selected for laboratory testing that consisted of moisture content determinations. The tests were performed in general accordance with test methods of the American Society for Testing and Materials (ASTM) or other applicable procedures.

Soil Classifications

All soil samples obtained from the explorations were visually classified in the field and/or in our laboratory using a system based on the Unified Soil Classification System (USCS) and ASTM classification methods. ASTM test method D 2488 was used to visually classify the soil samples, while ASTM D 2487 was used to classify the soils based on laboratory tests results. These classification procedures are incorporated in the exploration logs shown in Figure A-2 through A-20.

Moisture Content Determinations

Moisture contents were determined in general accordance with ASTM D 2216 for numerous samples obtained from the explorations. The results of these tests are also presented on the exploration logs at the respective sample depth in Appendix A.

SOIL CLASSIFICATION CHART

	A IOD DIVISI	ONE	SYMI	BOLS	TYPICAL	
IV	AJOR DIVISI	UNS	GRAPH LETTER		DESCRIPTIONS	
	GRAVEL	CLEAN GRAVELS		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES	
	AND GRAVELLY SOILS	(LITTLE OR NO FINES)		GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES	
COARSE GRAINED	MORE THAN 50% OF COARSE	GRAVELS WITH FINES		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES	
SOILS	FRACTION RETAINED ON NO. 4 SIEVE	(APPRECIABLE AMOUNT OF FINES)		GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES	
	SAND	CLEAN SANDS		sw	WELL-GRADED SANDS, GRAVELLY SANDS	
MORE THAN 50% RETAINED ON NO. 200 SIEVE	AND SANDY SOILS	(LITTLE OR NO FINES)		SP	POORLY-GRADED SANDS, GRAVELLY SAND	
	MORE THAN 50% OF COARSE	SANDS WITH FINES		SM	SILTY SANDS, SAND - SILT MIXTURES	
	PASSING NO. 4 SIEVE	(APPRECIABLE AMOUNT OF FINES)		sc	CLAYEY SANDS, SAND - CLAY MIXTURES	
				ML	INORGANIC SILTS, ROCK FLOUR, CLAYEY SILTS WITH SLIGHT PLASTICITY	
FINE GRAINED	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS	
SOILS				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY	
MORE THAN 50% PASSING NO. 200			Ţij	МН	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS SILTY SOILS	
SIEVE	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		СН	INORGANIC CLAYS OF HIGH PLASTICITY	
			hil	ОН	ORGANIC CLAYS AND SILTS OF MEDIUM TO HIGH PLASTICITY	
н	GHLY ORGANIC	SOILS	FF	PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS	

NOTE: Multiple symbols are used to indicate borderline or duel soil classifications

Sampler Symbol Descriptions

2.4-inch I.D. split barrel

Standard Penetration Test (SPT)

Shelby tube

Piston
Direct-Push

Bulk or grab

Blowcount is recorded for driven samplers as the number of blows required to advance sampler 12 inches (or distance noted). See exploration log for hammer weight and drop.

 \boldsymbol{A} "P" indicates sampler pushed using the weight of the drill rig.

ADDITIONAL MATERIAL SYMBOLS

SYMI	BOLS	TYPICAL
GRAPH	LETTER	DESCRIPTIONS
	СС	Cement Concrete
	AC	Asphalt
	CR	Crushed Rock/ Quarry Spalls
	TS	Topsoil/ Forest Duff/Sod



Measured groundwater level



Groundwater observed at time of exploration



Perched water observed at time of exploration



Measured free product in well or piezometer

Stratigraphic Contact

Distict contact between soil strata or geologic units
Gradual change between soil strata or

geologic units

Approximate location of soil strata change within a geologic soil unit

Laboratory / Field Tests

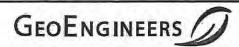
Percent fines AL CA CP Atterberg limits Chemical analysis Laboratory compaction test CS Consolidation test DS Direct shear HA Hydrometer analysis MC Moisture content MD Moisture content and dry density OC Organic content PM Permeability or hydraulic conductivity Pocket penetrometer PP SA Sieve analysis TX Triaxial compression UC Unconfined compression Vane shear

Sheen Classification

NS No Visible Sheen SS Slight Sheen MS Moderate Sheen HS Heavy Sheen NT Not Tested

NOTE: The reader must refer to the discussion in the report text and the logs of explorations for a proper understanding of subsurface conditions. Descriptions on the logs apply only at the specific exploration locations and at the time the explorations were made; they are not warranted to be representative of subsurface conditions at other locations or times.

KEY TO EXPLORATION LOGS



Date Excavated: 07/12/04

Logged by: ______MET

Equipment: Case 580 Rubber-tired Backhoe

Surface Elevation (ft): Approximately 106

feet	Depth	ample	Sample Number	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	OTHER TESTS AND NOTES
	0-	Š	Ö	ω	U O	Topsoil/root zone	≥0	
					SM	Brown silty fine to medium sand with occasional gravel (medium dense, moist) (weathered till)		
		M	-1		SM	Grayish brown silty fine to medium sand with gravel (dense, moist) (till)		
	5-				ML	Brown sandy silt (very stiff, moist) (transitional beds)		
					-			
	-	X	2		+		1	
	-	M	3					
	10-					Test pit completed at 9.5 feet on 07/12/04 No groundwater seepage observed No caving observed		
	-							
	_					*		
	15-					nation of symbols. ure based on an average of measurements across the test pit and should be con		

LOG OF TEST PIT TP-1



Project:

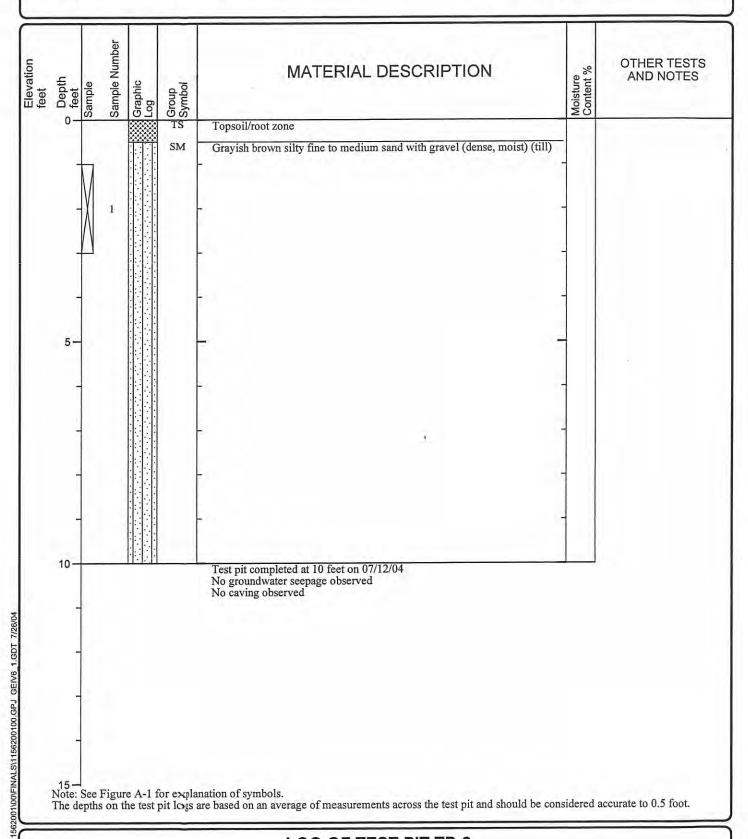
Washington Cathedral

Project Location: Redmond, Washington

Project Number: 11562-001-00

FIGURE A-2 Sheet 1 of 1 Date Excavated: 07/12/04 Logged by: MET

Equipment: Case 580 Rubber-tired Backhoe Surface Elevation (ft): Approximately 153



LOG OF TEST PIT TP-2



Project: Washington Cathedral Project Location: Redmond, Washington

Project Number: 11562-001-00

FIGURE A-3 Sheet 1 of 1

07/12/04 Date Excavated: _____

KHC Logged by: ____

Equipment: Case 580 Rubber-tired Backhoe

Surface Elevation (ft): Approximately 143

feet	Depth feet Sample	Sample Number	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	OTHER TESTS AND NOTES
		i		TS SM	Topsoil/root zone Grayish brown sandy silt (medium stiff, moist) (fill) -		
		2		SM	Grayish brown silty fine to medium sand with gravel (dense, moist) (till)	-	
	5	3		ML	Grayish brown sandy silt (very stiff, moist) (native) (transitional beds)	-	
							1
	10-				Test pit completed at 9 feet on 07/12/04 No groundwater seepage observed No caving observed		
	-						

LOG OF TEST PIT TP-3



Project:

Washington Cathedral

Project Location: Redmond, Washington

Project Number: 11562-001-00

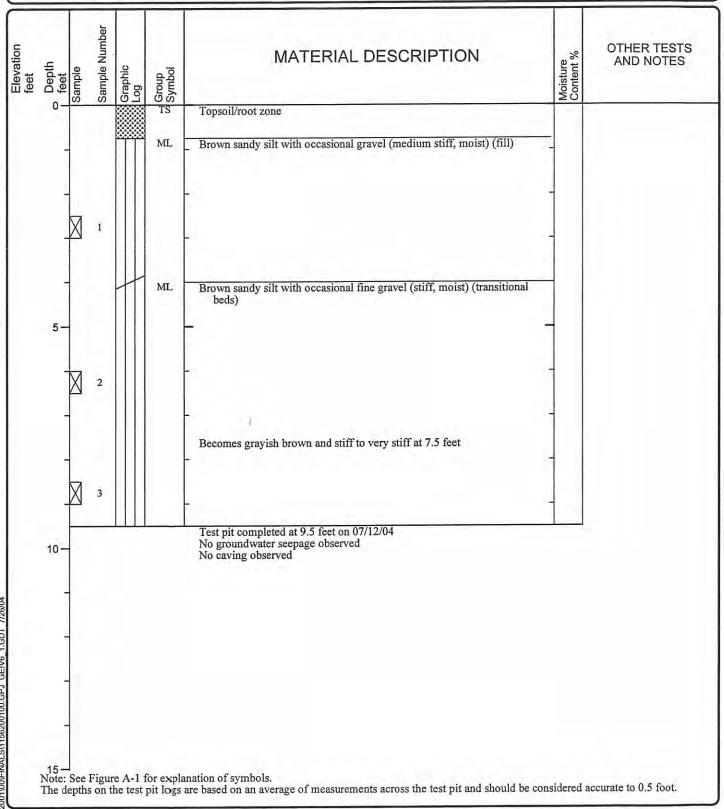
FIGURE A-4 Sheet 1 of 1

07/12/04 Date Excavated: _____

KHC Logged by: _____

Equipment: Case 580 Rubber-tired Backhoe

Surface Elevation (ft): Approximately 115



LOG OF TEST PIT TP-4 Washington Cathedral Project: GEOENGINEERS /

Project Location: Redmond, Washington

FIGURE A-5 Sheet 1 of 1

Project Number: 11562-001-00

Date Excavated: 07/12/04 Logged by: KHC

Equipment: Case 580 Rubber-tired Backhoe Surface Elevation (ft): Approximately 156

feet	Depth feet	Sample	Sample Number	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	OTHER TESTS AND NOTES
	0-		4		TS	Topsoil/root zone		
		Θ	1 2		ML	Grayish brown sandy silt with gravel (stiff, moist) (fill)		
		M	<	M	OL	Dark brown organic silt (soft, moist) (old topsoil)		
					SM	Grayish brown silty fine to medium sand with gravel (dense to very dense, moist) (till)		
	Ī	M	3				15	
	-					· 1		
	5-					7		
	_							
	9	X	4					
	-					3		
	-					Test pit completed at 9 feet on 07/12/04 No groundwater seepage observed No caving observed		
	10-					No caving observed		
	4							
	_							
						anation of symbols. are based on an average of measurements across the test pit and should be cons		

LOG OF TEST PIT TP-5

Project Number: 11562-001-00



Project: Washington Cathedral Project Location: Redmond, Washington

FIGURE A-6 Sheet 1 of 1 Date Excavated: _____07/12/04

Logged by: KHC

Equipment: Case 580 Rubber-tired Backhoe

Surface Elevation (ft): Approximately 144

reet Depth	ample	Sample Number	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	OTHER TESTS AND NOTES
0-	S	S	W 2	TS	Topsoil/root zone	20	
				ML	Grayish brown sandy silt with gravel (medium stiff, moist) (fill)		
	M				*		
	$\frac{1}{1}$	1	Ш			-	
	\mathbb{N}		Ш				
	T						
	-				o contract the second s	-	
5-						7 1	
	$\frac{1}{1}$			ML	Gray silt with sand (soft, moist to wet) (fill)	4	
	X	2			Olay one want does not not come		
	1	(
				OL	Dark brown organic silt (soft, moist) (old topsoil)	4	
			1111				
	-			SM	Grayish brown silty fine to medium sand with gravel (dense to very dense moist) (till)	2,	
10							
10 -	M	3					
	1				Test pit completed at 11 feet on 07/12/04		
					No groundwater seepage observed No caving observed		
	1						
	-						
15							
15 - Note The	: See depth	Figur	re A-1 f	or expla	anation of symbols. are based on an average of measurements across the test pit and should be co	nsidered	accurate to 0.5 foot

LOG OF TEST PIT TP-6



Project: Washington Cathedral

Project Location: Redmond, Washington

Project Number: 11562-001-00

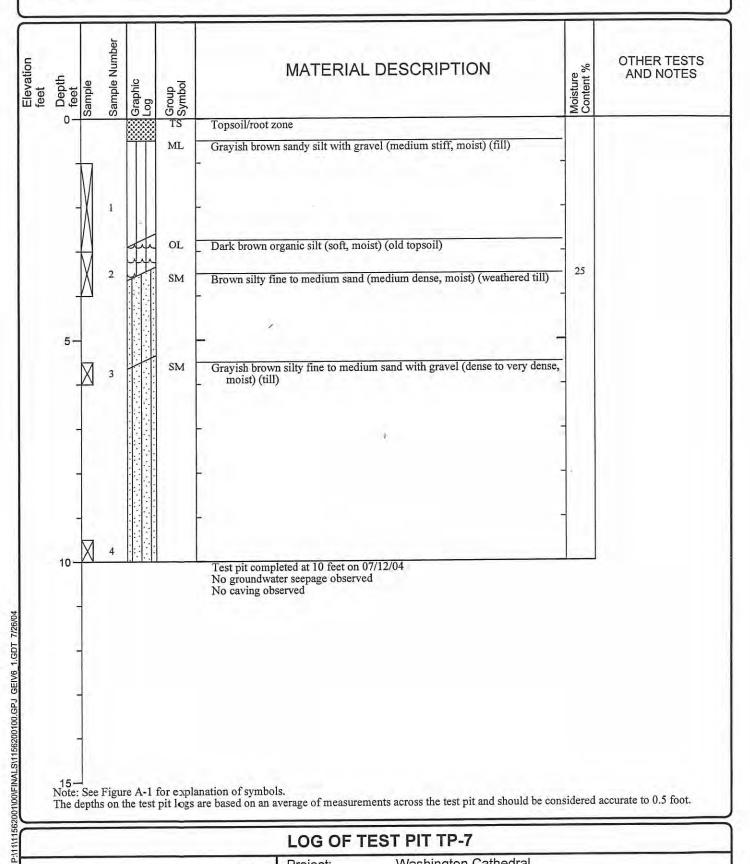
FIGURE A-7 Sheet 1 of 1

07/12/04 Date Excavated: _

Logged by: _____ KHC

Equipment: Case 580 Rubber-tired Backhoe

Surface Elevation (ft): Approximately 154



LOG OF TEST PIT TP-7



Project:

Washington Cathedral

Project Location: Redmond, Washington

Project Number: 11562-001-00

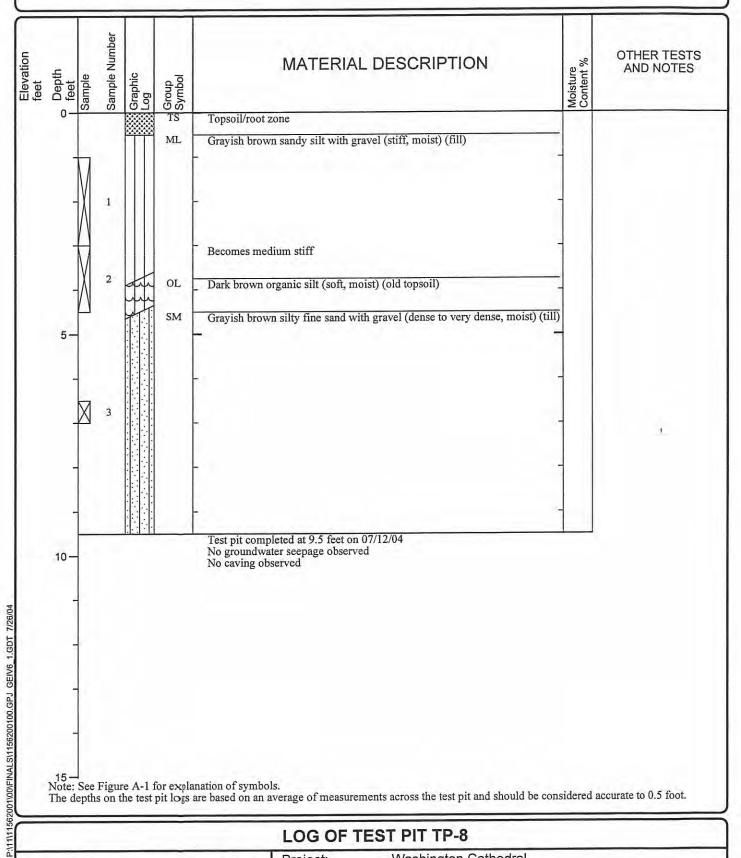
FIGURE A-8 Sheet 1 of 1

07/12/04 Date Excavated: ___

KHC Logged by: _____

Equipment: Case 580 Rubber-tired Backhoe

Surface Elevation (ft): Approximately 143



LOG OF TEST PIT TP-8



Project:

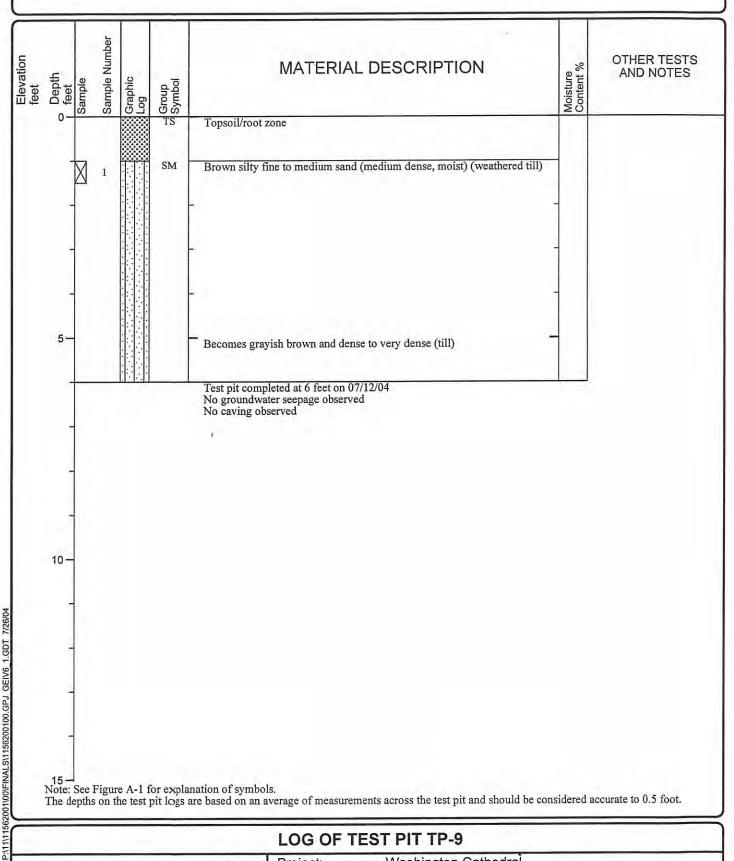
Washington Cathedral

Project Location: Redmond, Washington

Project Number: 11562-001-00

FIGURE A-9 Sheet 1 of 1

Date Excavated: 07/12/04 Logged by: _____ Surface Elevation (ft): Approximately 169 Equipment: Case 580 Rubber-tired Backhoe



LOG OF TEST PIT TP-9



Washington Cathedral Project:

Project Number: 11562-001-00

Project Location: Redmond, Washington

FIGURE A-10 Sheet 1 of 1

Date Excavated: ____

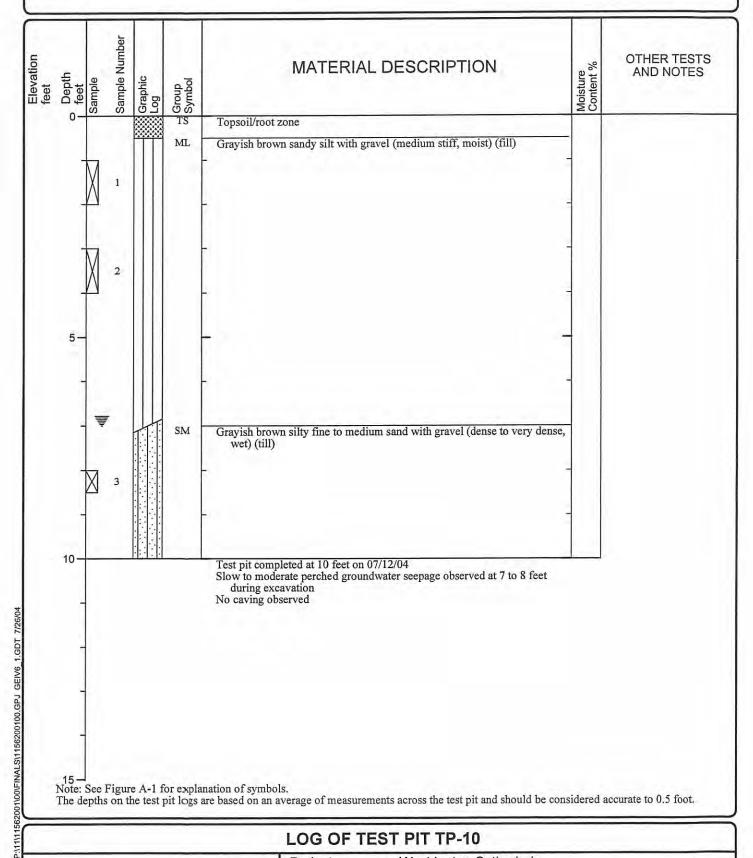
07/12/04

Logged by: __

Equipment: Case 580 Rubber-tired Backhoe

Surface Elevation (ft): Approximately 143

KHC



LOG OF TEST PIT TP-10



Project:

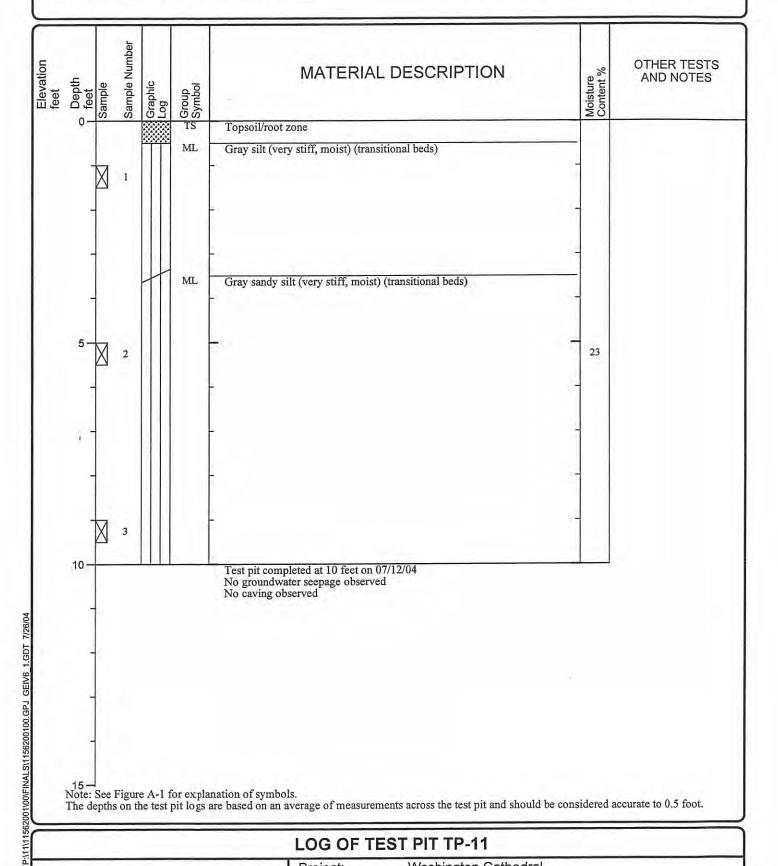
Washington Cathedral

Project Location: Redmond, Washington

Project Number: 11562-001-00

FIGURE A-11 Sheet 1 of 1

Date Excavated: ____ 07/12/04 Logged by: ____ Surface Elevation (ft): Approximately 103 Equipment: Case 580 Rubber-tired Backhoe



LOG OF TEST PIT TP-11



Project:

Washington Cathedral

Project Location: Redmond, Washington

Project Number: 11562-001-00

FIGURE A-12 Sheet 1 of 1

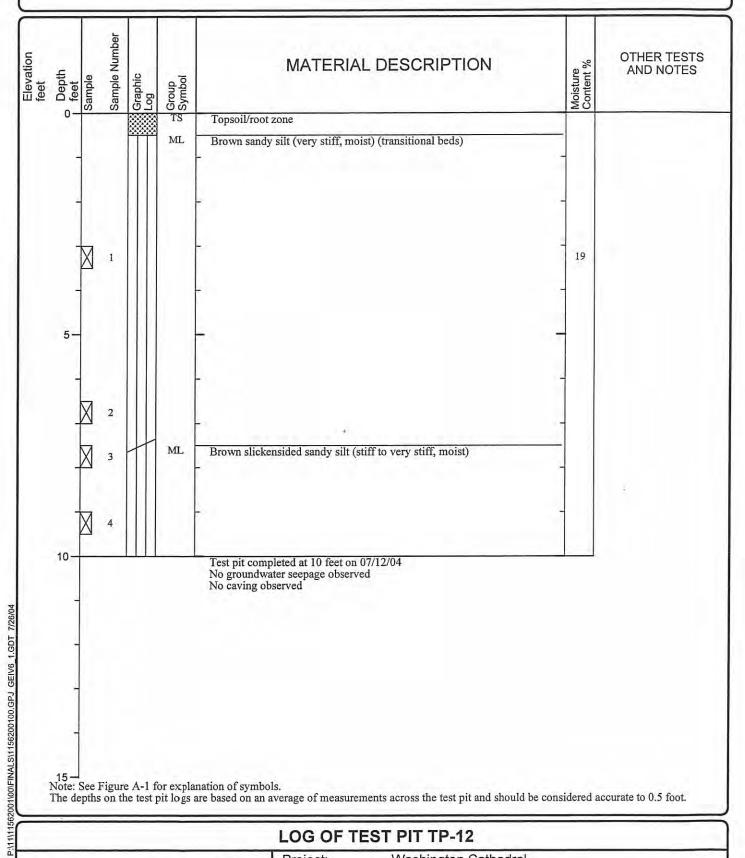
MET

07/12/04 Date Excavated: ___

MET Logged by: _____

Equipment: Case 580 Rubber-tired Backhoe

Surface Elevation (ft): Approximately 101



LOG OF TEST PIT TP-12



Project:

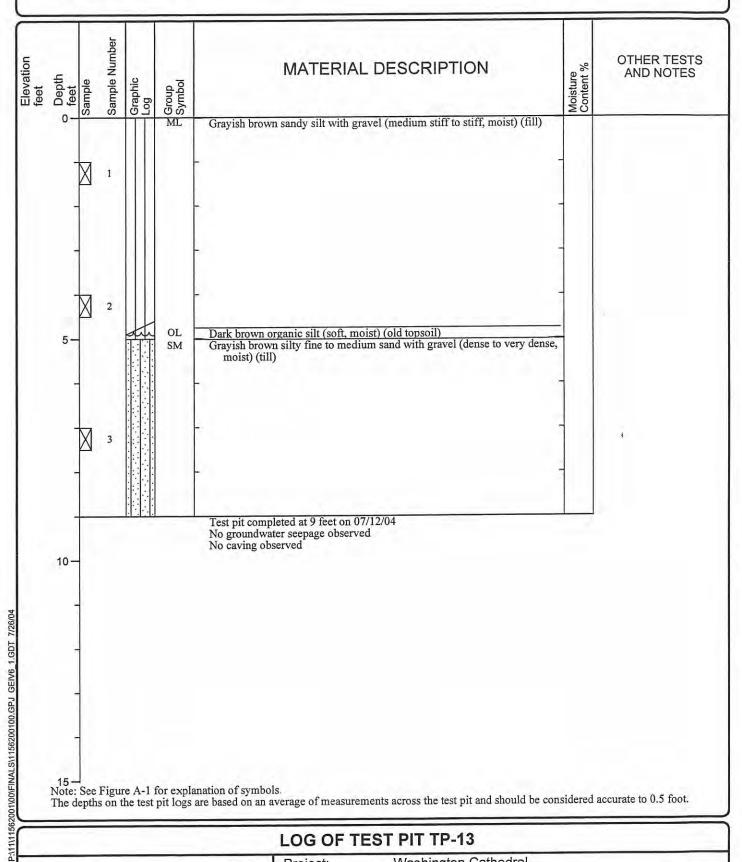
Washington Cathedral

Project Location: Redmond, Washington

Project Number: 11562-001-00

FIGURE A-13 Sheet 1 of 1

MET 07/12/04 Logged by: _____ Date Excavated: _____ Surface Elevation (ft): Approximately 135 Equipment: Case 580 Rubber-tired Backhoe



LOG OF TEST PIT TP-13



Washington Cathedral Project: Project Location: Redmond, Washington

Project Number: 11562-001-00

FIGURE A-14 Sheet 1 of 1

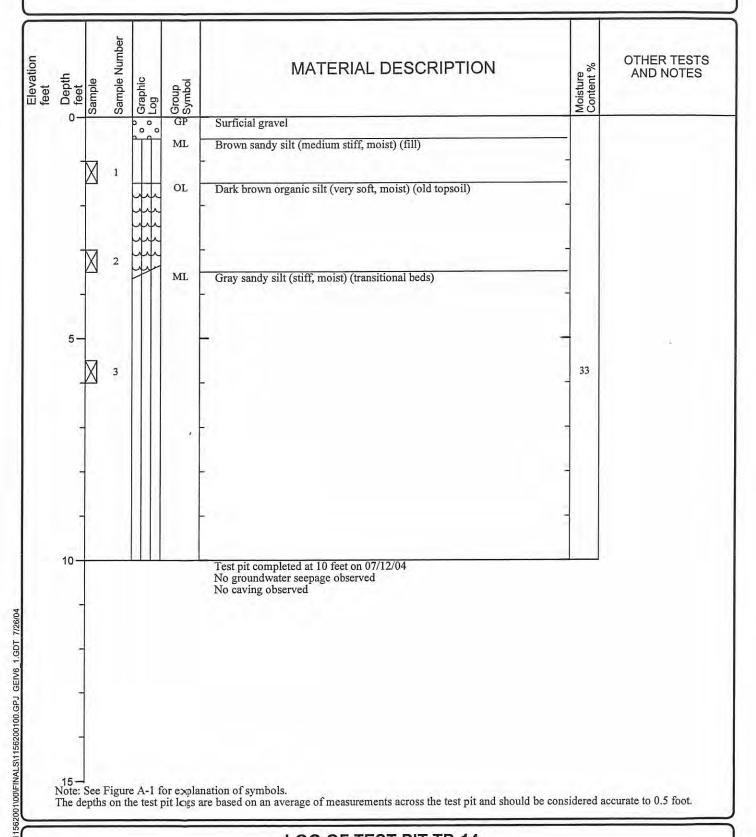
Date Excavated: _____

07/12/04

MET Logged by: __

Equipment: Case 580 Rubber-tired Backhoe

Surface Elevation (ft): Approximately 108



LOG OF TEST PIT TP-14



Project:

Washington Cathedral

Project Location: Redmond, Washington

Project Number: 11562-001-00

FIGURE A-15

Sheet 1 of 1

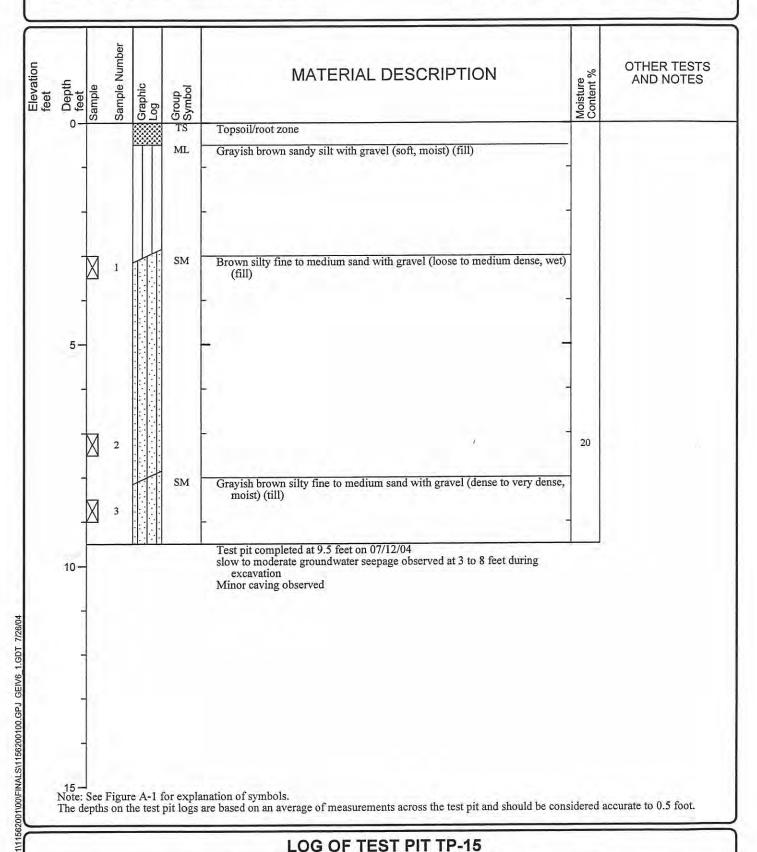
Date Excavated: 07/12/04

Logged by: _____

Equipment: Case 580 Rubber-tired Backhoe

Surface Elevation (ft): Approximately 131

MET



Washington Cathedral Project: GEOENGINEERS /

Project Location: Redmond, Washington Project Number: 11562-001-00

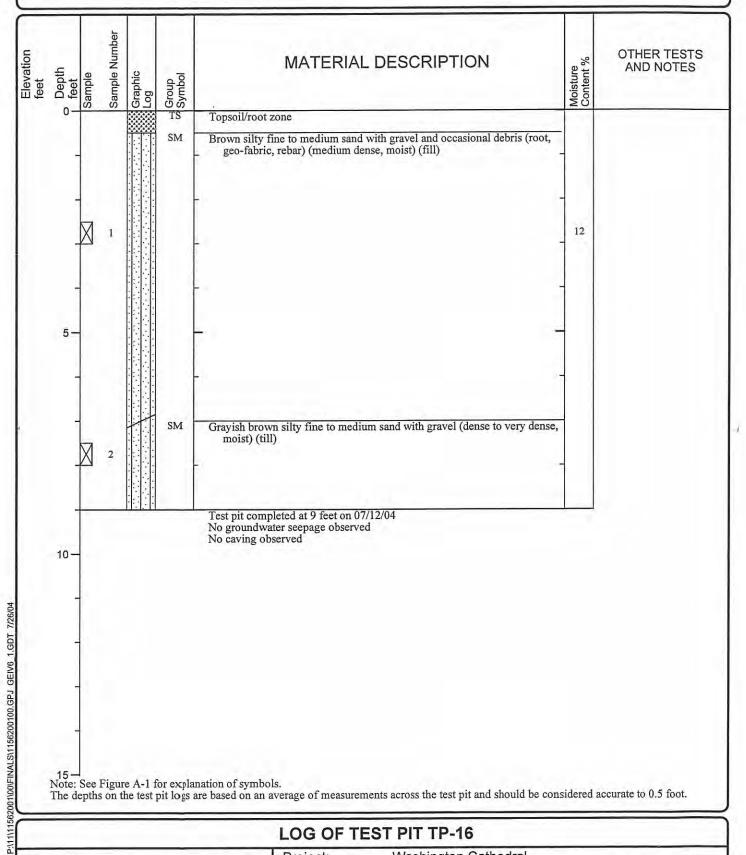
FIGURE A-16 Sheet 1 of 1

07/12/04 Date Excavated: _

MET Logged by: _____

Equipment: Case 580 Rubber-tired Backhoe

Surface Elevation (ft): Approximately 115



LOG OF TEST PIT TP-16



Project:

Washington Cathedral

Project Location: Redmond, Washington

Project Number: 11562-001-00

FIGURE A-17 Sheet 1 of 1

Date Excavated: _

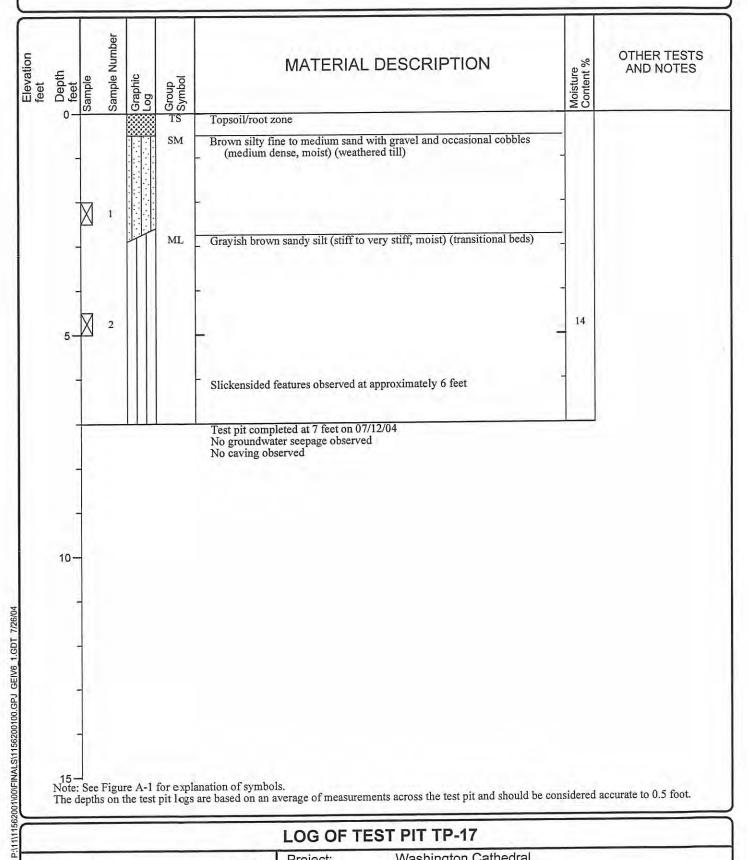
07/12/04

Logged by: _____

MET

Equipment: Case 580 Rubber-tired Backhoe

Surface Elevation (ft): Approximately 99



LOG OF TEST PIT TP-17



Project:

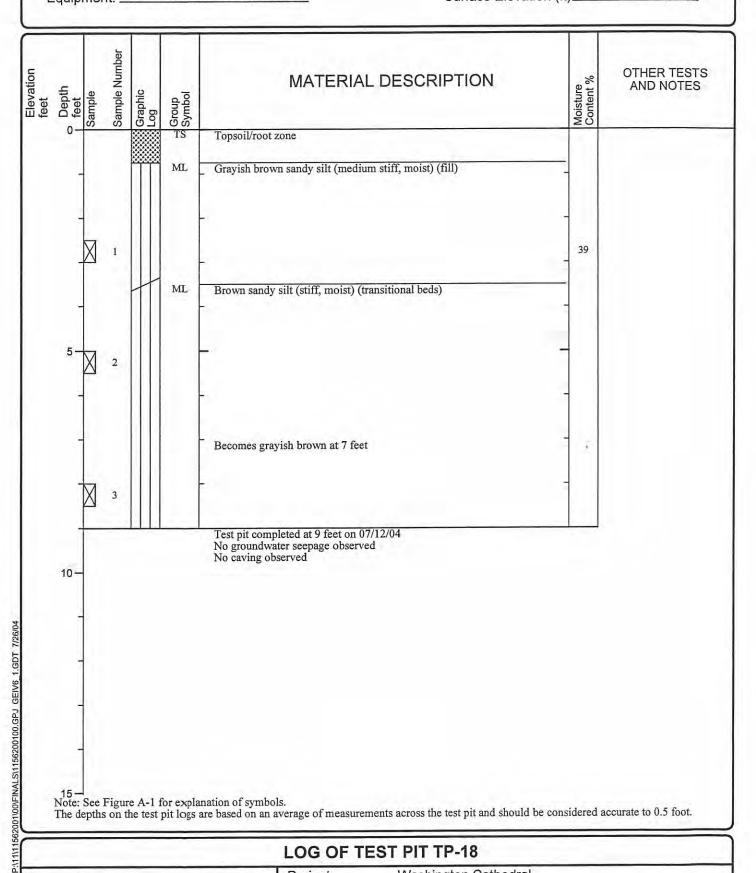
Washington Cathedral

Project Location: Redmond, Washington

Project Number: 11562-001-00

FIGURE A-18 Sheet 1 of 1

07/12/04 Date Excavated: ____ Logged by: _____ Surface Elevation (ft): Approximately 66 Equipment: Case 580 Rubber-tired Backhoe



Project: GEOENGINEERS /

Washington Cathedral Project Location: Redmond, Washington

FIGURE A-19 Project Number: 11562-001-00

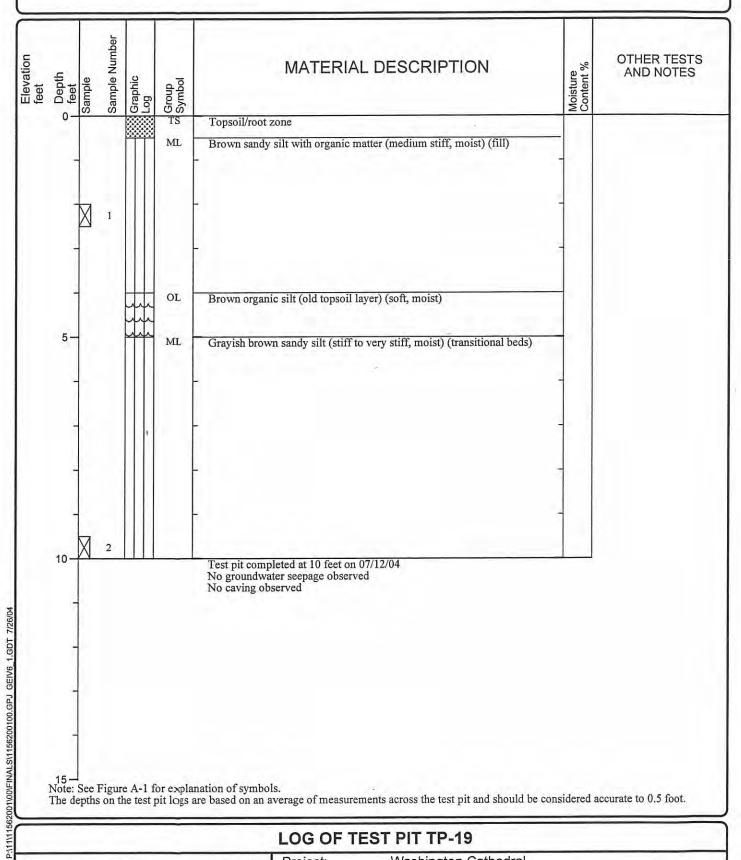
Sheet 1 of 1

Date Excavated: _____07/12/04

Logged by: _____

Equipment: Case 580 Rubber-tired Backhoe

Surface Elevation (ft): Approximately 60



LOG OF TEST PIT TP-19



Washington Cathedral Project:

Project Location: Redmond, Washington

Project Number: 11562-001-00

FIGURE A-20 Sheet 1 of 1



APPENDIX B REPORT LIMITATIONS AND GUIDELINES FOR USE

APPENDIX B

REPORT LIMITATIONS AND GUIDELINES FOR USE1

This appendix provides information to help you manage your risks with respect to the use of this report.

GEOTECHNICAL SERVICES ARE PERFORMED FOR SPECIFIC PURPOSES, PERSONS AND PROJECTS

This report has been prepared for use by Washington Cathedral, Fourfold Architecture PLLC, and members of the design team for use in the design of this project. This report may be made available to prospective contractors for bidding or estimating purposes; but our report, conclusions and interpretations should not be construed as a warranty of the subsurface conditions. This report is not intended for use by others, and the information contained herein is not applicable to other sites.

GeoEngineers structures our services to meet the specific needs of our clients. For example, a geotechnical or geologic study conducted for a civil engineer or architect may not fulfill the needs of a construction contractor or even another civil engineer or architect that are involved in the same project. Because each geotechnical or geologic study is unique, each geotechnical engineering or geologic report is unique, prepared solely for the specific client and project site. No one except Everett Community College, LMN Architects, and members of the design team should rely on this report without first conferring with GeoEngineers. This report should not be applied for any purpose or project except the one originally contemplated.

A GEOTECHNICAL ENGINEERING OR GEOLOGIC REPORT IS BASED ON A UNIQUE SET OF PROJECT-SPECIFIC FACTORS

This report has been prepared for the proposed gymnasium complex building and community center that will be located at Washington Cathedral property in Redmond, Washington. GeoEngineers considered a number of unique, project-specific factors when establishing the scope of services for this project and report. Unless GeoEngineers specifically indicates otherwise, do not rely on this report if it was:

- not prepared for you,
- not prepared for your project,
- · not prepared for the specific site explored, or
- · completed before important project changes were made.

For example, changes that can affect the applicability of this report include those that affect:

- the function of the proposed structure;
- elevation, configuration, location, orientation or weight of the proposed structure;
- · composition of the design team; or
- project ownership.

¹ Developed based on material provided by ASFE, Professional Firms Practicing in the Geosciences; www.asfe.org.

If important changes are made after the date of this report, GeoEngineers should be given the opportunity to review our interpretations and recommendations and provide written modifications or confirmation, as appropriate.

SUBSURFACE CONDITIONS CAN CHANGE

This geotechnical or geologic report is based on conditions that existed at the time the study was performed. The findings and conclusions of this report may be affected by the passage of time, by manmade events such as construction on or adjacent to the site, or by natural events such as floods, earthquakes, slope instability or groundwater fluctuations. Always contact GeoEngineers before applying a report to determine if it remains applicable.

MOST GEOTECHNICAL AND GEOLOGIC FINDINGS ARE PROFESSIONAL OPINIONS

Our interpretations of subsurface conditions are based on field observations from widely spaced sampling locations at the site. Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. GeoEngineers reviewed field and laboratory data and then applied our professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ, sometimes significantly, from those indicated in this report. Our report, conclusions and interpretations should not be construed as a warranty of the subsurface conditions.

GEOTECHNICAL ENGINEERING REPORT RECOMMENDATIONS ARE NOT FINAL

Do not over-rely on the preliminary construction recommendations included in this report. These recommendations are not final, because they were developed principally from GeoEngineers' professional judgment and opinion. GeoEngineers' recommendations can be finalized only by observing actual subsurface conditions revealed during construction. GeoEngineers cannot assume responsibility or liability for this report's recommendations if we do not perform construction observation.

Sufficient monitoring, testing and consultation by GeoEngineers should be provided during construction to confirm that the conditions encountered are consistent with those indicated by the explorations, to provide recommendations for design changes should the conditions revealed during the work differ from those anticipated, and to evaluate whether or not earthwork activities are completed in accordance with our recommendations. Retaining GeoEngineers for construction observation for this project is the most effective method of managing the risks associated with unanticipated conditions.

A GEOTECHNICAL ENGINEERING OR GEOLOGIC REPORT COULD BE SUBJECT TO MISINTERPRETATION

Misinterpretation of this report by other design team members can result in costly problems. You could lower that risk by having GeoEngineers confer with appropriate members of the design team after submitting the report. Also retain GeoEngineers to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering or geologic report. Reduce that risk by having GeoEngineers participate in pre-bid and preconstruction conferences, and by providing construction observation.

DO NOT REDRAW THE EXPLORATION LOGS

Geotechnical engineers and geologists prepare final test pit and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering or geologic report should never be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, but recognize that separating logs from the report can elevate risk.

GIVE CONTRACTORS A COMPLETE REPORT AND GUIDANCE

Some owners and design professionals believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering or geologic report, but preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with GeoEngineers and/or to conduct additional study to obtain the specific types of information they need or prefer. A pre-bid conference can also be valuable. Be sure contractors have sufficient time to perform additional study. Only then might an owner be in a position to give contractors the best information available, while requiring them to at least share the financial responsibilities stemming from unanticipated conditions. Further, a contingency for unanticipated conditions should be included in your project budget and schedule.

CONTRACTORS ARE RESPONSIBLE FOR SITE SAFETY ON THEIR OWN CONSTRUCTION PROJECTS

Our geotechnical recommendations are not intended to direct the contractor's procedures, methods, schedule or management of the work site. The contractor is solely responsible for job site safety and for managing construction operations to minimize risks to on-site personnel and to adjacent properties.

READ THESE PROVISIONS CLOSELY

Some clients, design professionals and contractors may not recognize that the geoscience practices (geotechnical engineering or geology) are far less exact than other engineering and natural science disciplines. This lack of understanding can create unrealistic expectations that could lead to disappointments, claims and disputes. GeoEngineers includes these explanatory "limitations" provisions in our reports to help reduce such risks. Please confer with GeoEngineers if you are unclear how these "Report Limitations and Guidelines for Use" apply to your project or site.

GEOTECHNICAL, GEOLOGIC AND ENVIRONMENTAL REPORTS SHOULD NOT BE INTERCHANGED

The equipment, techniques and personnel used to perform an environmental study differ significantly from those used to perform a geotechnical or geologic study and vice versa. For that reason, a geotechnical engineering or geologic report does not usually relate any environmental findings, conclusions or recommendations; e.g., about the likelihood of encountering underground storage tanks or

regulated contaminants. Similarly, environmental reports are not used to address geotechnical or geologic concerns regarding a specific project.

BIOLOGICAL POLLUTANTS

2 0

GeoEngineers' Scope of Work specifically excludes the investigation, detection, prevention, or assessment of the presence of Biological Pollutants in or around any structure. Accordingly, this report includes no interpretations, recommendations, findings, or conclusions for the purpose of detecting, preventing, assessing, or abating Biological Pollutants. The term "Biological Pollutants" includes, but is not limited to, molds, fungi, spores, bacteria, and viruses, and/or any of their byproducts.



APPENDIX C

	CONVEYANCE SYSTEM ANALYSIS AND SIZING TABLE USING THE RATIONAL METHOD																		
Lo	Location Sub Area C C*A Sum Tc i(R) I(R) Q(R) Pipe Typ. Slope Q(F) V V L Tt % d/D																		
		Basin	(ac)			C*A	(min.)			(c.f.s.)	(in.)	n	(ft.ft.)	(pipe	(pipe	(at	(ft.)	(min.)	
From	To	Number												full)	full)	Q(R))			
		All	1.17	0.73	0.85	0.85	6.3	0.80	2.53	2.16	12	0.012	0.0100	3.87	4.93	4.99	100.0	0.3	54.00%
Project																			
Locatio	ocation: Date: 2/1/2017 Page 1 of 1																		

"C" VALUE (from King County 2016 SWDM Table 3.2.1.A)

	Area (Sq.	Area	С
Landcover	Ft.)	(Acres)	Value
Impervious	37,480	0.86	0.9
Lawns	13470	0.31	0.25

Composite C Value

CHECK:

Peak Runoff Rate \leq Pipe Capacity $Q(R) \leq Q(F)$ 2.16 \leq 3.87 True

2/1/2017 C-1 Rational Method.xls

FIGURE 3.2.1.C 25-YEAR 24-HOUR ISOPLUVIALS 2.5 KING COUNTY 日 2.9 0 ் ம 0 Puget So **@** Φjø KING COUNTY **WESTERN** KING COUNTY 25-Year 24-Hour 2. 0. 0. 0. **Precipitation** 4 Miles in Inches

WWHM2012 PROJECT REPORT

Project Name: Phase 1 Flow Comparison

Site Name: Site Address: City :

Report Date: 2/2/2017
Gage : Seatac

Data Start : 1948/10/01 Data End : 2009/09/30 Precip Scale: 1.00

Version Date: 2016/02/25

Version : 4.2.12

Low Flow Threshold for POC 1: 50 Percent of the 2 Year

High Flow Threshold for POC 1: 50 year

PREDEVELOPED LAND USE

Name : Basin 1

Bypass: No

GroundWater: No

Perv	vious	Land	Use	acre
C,	Lawn	, Flat		.13

Pervious Total 0.13

Impervious Land Use	acre
ROADS FLAT	0.685
ROOF TOPS FLAT	0.305
SIDEWALKS FLAT	0.06

Impervious Total 1.05

Basin Total 1.18

Element Flows To:

Surface Interflow Groundwater

MITIGATED LAND USE

Name : Basin 1

Bypass: No

GroundWater: No

Pervious Land Use C, Lawn, Flat	<u>acre</u> .31
Pervious Total	0.31
Impervious Land Use	acre

Impervious Land Use	acre
ROADS FLAT	0.48
ROOF TOPS FLAT	0.23
SIDEWALKS FLAT	0.16
Impervious Total	0.87
Basin Total	1.18

Element Flows To:

Surface Interflow Groundwater

ANALYSIS RESULTS

Stream Protection Duration

Predeveloped Landuse Totals for POC #1

Total Pervious Area:0.13
Total Impervious Area:1.05

Mitigated Landuse Totals for POC #1

Total Pervious Area:0.31
Total Impervious Area:0.87

Flow Frequency Return Periods for Predeveloped. POC #1

Return Period	Flow(cfs)
2 year	0.274982
5 year	0.346894
10 year	0.396706
25 year	0.462305
50 year	0.513188
100 year	0.565863

Flow Frequency Return Periods for Mitigated. POC #1

Return Period	Flow(cfs)
2 year	0.237894
5 year	0.303288

10 year	0.348958
25 year	0.409496
50 year	0.456726
100 year	0.505844

Stream Protection Duration

Annual Peaks for Predeveloped and Mitigated. POC #1

Annual	Peaks	for	Predeveloped	d and	Mitigated.	PC
Year		Pred	developed	Miti	gated	
1949		0	.276	0.2	41	
1950		0	.403	0.3	67	
1951		0	. 269	0.2	40	
1952		0	.236	0.1	99	
1953			.214	0.1		
1954			.256	0.2		
1955			. 273	0.2		
1956			.263	0.2		
1957			.301	0.2		
1958			.271	0.2		
1959			.208	0.1		
1960			.261	0.2		
1961			.227	0.1		
1962			.233	0.1		
1963			. 223	0.1		
1964			.264	0.2		
1965			.236	0.2		
1966			.234	0.2		
1967			.353	0.3		
1968			.397	0.3		
1969			.214	0.1		
1970			.235	0.2		
1971			. 227	0.1		
1972			.328	0.2		
1973			.214	0.1		
1974			.236	0.2		
1975			.312	0.2		
1976			.214	0.1		
1977			.271	0.2		
1978			.350	0.2		
1979			.346	0.2		
1980			.291	0.2		
1981			.323	0.2		
1982			.442	0.3		
1983			.322	0.2		
1984			.244	0.2		
1985			.230	0.1		
1986			.287	0.2		
1987			.401	0.3		
1988			.191	0.1		
1989			.240	0.1		
1990			.451	0.4		
1991			.421	0.3		
1992			. 242	0.2		
1993			.166	0.1		
1994			.198	0.1		
1995			. 249	0.2		
1996			. 296	0.2		

1997	0.284	0.251
1998	0.306	0.264
1999	0.578	0.503
2000	0.297	0.258
2001	0.254	0.212
2002	0.298	0.261
2003	0.269	0.235
2004	0.520	0.463
2005	0.244	0.210
2006	0.202	0.179
2007	0.464	0.433
2008	0.415	0.373
2009	0.287	0.244

Stream Protection Duration

Ranked Annual Peaks for Predeveloped and Mitigated. POC #1

ramea	IIIIIdai I Canb IOI	TICACVCIOPCA AMA	mile in garca.	100 11-
Rank	Predeveloped	Mitigated		
1	0.5782	0.5034		
2	0.5202	0.4627		
3	0.4645	0.4328		
4	0.4509	0.4084		
5	0.4420	0.3861		
6	0.4210	0.3774		
7	0.4153	0.3726		
8	0.4032	0.3672		
9	0.4011	0.3403		
10	0.3975	0.3377		
11	0.3527	0.3041		
12	0.3501	0.2937		
13	0.3456	0.2894		
14	0.3280	0.2889		
15	0.3226	0.2787		
16	0.3220	0.2760		
17	0.3125	0.2725		
18	0.3063	0.2665		
19	0.3010	0.2643		
20	0.2975	0.2635		
21	0.2974	0.2610		
22	0.2956	0.2581		
23	0.2914	0.2549		
24	0.2873	0.2548		
25	0.2866	0.2508		
26	0.2839	0.2441		
27	0.2762	0.2414		
28	0.2733	0.2401		
29	0.2714	0.2353		
30	0.2707	0.2348		
31	0.2694	0.2337		
32	0.2686	0.2291		
33	0.2644	0.2279		
34	0.2630	0.2277		
35	0.2607	0.2254		
36	0.2557	0.2214		
37	0.2543	0.2124		
38	0.2488	0.2120		
39	0.2440	0.2110		

40 41 42 43 44 45 46 47 48 49 50 51 52 53 54 55 56	0.2436 0.2425 0.2403 0.2363 0.2358 0.2356 0.2348 0.2345 0.2328 0.2304 0.2275 0.2272 0.2231 0.2141 0.2140 0.2137 0.2137 0.2081 0.2017	0.2102 0.2090 0.2050 0.2029 0.2026 0.2023 0.1991 0.1990 0.1979 0.1970 0.1951 0.1937 0.1936 0.1866 0.1859 0.1844 0.1827 0.1792
57	0.2081	0.1792
58	0.2017	0.1788
59	0.1977	0.1655
60	0.1914	0.1586
61	0.1661	0.1403

Stream Protection Duration POC #1
The Facility PASSED

The Facility PASSED.

Flow(cfs)	Predev	Mit	Percentage	e Pass/Fail
0.1375	1324	790	59	Pass
0.1413	1200	713	59	Pass
0.1451	1086	648	59	Pass
0.1489	1020	598	58	Pass
0.1527	934	552	59	Pass
0.1565	850	513	60	Pass
0.1603	788	457	57	Pass
0.1641	724	426	58	Pass
0.1679	677	383	56	Pass
0.1716	623	355	56	Pass
0.1754	576	323	56	Pass
0.1792	537	301	56	Pass
0.1830	492	273	55	Pass
0.1868	461	259	56	Pass
0.1906	427	244	57	Pass
0.1944	395	222	56	Pass
0.1982	358	204	56	Pass
0.2020	324	189	58	Pass
0.2058	309	180	58	Pass
0.2096	289	168	58	Pass
0.2134	272	151	55	Pass
0.2172	259	140	54	Pass
0.2210	238	136	57	Pass
0.2248	221	128	57	Pass
0.2286	208	116	55	Pass
0.2324	197	109	55	Pass

0.2362	182	100	54	Pass
0.2400	172	93	54	Pass
0.2437	158	81	51	Pass
0.2475	153	77 71	50	Pass
0.2513	142	71	50	Pass
0.2551 0.2589	134 129	68 65	50 50	Pass
0.2569	129	61	50	Pass
0.2665	109	57	52	Pass Pass
0.2703	104	56	53	Pass
0.2703	91	54	59	Pass
0.2711	86	52	60	Pass
0.2817	84	50	59	Pass
0.2855	78	48	61	Pass
0.2893	72	44	61	Pass
0.2931	69	41	59	Pass
0.2969	65	38	58	Pass
0.3007	60	38	63	Pass
0.3045	58	35	60	Pass
0.3083	55	31	56	Pass
0.3121	53	31	58	Pass
0.3159	52	31	59	Pass
0.3196	52	29	55	Pass
0.3234	49	26	53	Pass
0.3272	47	26	55	Pass
0.3310	45	24	53	Pass
0.3348	43	24	55	Pass
0.3386	41	22	53	Pass
0.3424	39	19	48	Pass
0.3462	35	19	54	Pass
0.3500	34	19	55	Pass
0.3538	30	16	53	Pass
0.3576	30	15	50	Pass
0.3614	28	14	50	Pass
0.3652	27	14	51	Pass
0.3690	27	12	44	Pass
0.3728	25	11 10	44	Pass
0.3766	24		41	Pass
0.3804 0.3842	24 22	8 8	33 36	Pass
0.3842	21	7	33	Pass Pass
0.3918	21	7	33	Pass
0.3955	19	6	31	Pass
0.3993	17	6	35	Pass
0.4031	16	6	37	Pass
0.4069	14	6	42	Pass
0.4107	12	5	41	Pass
0.4145	12	5	41	Pass
0.4183	10	5	50	Pass
0.4221	9	5	55	Pass
0.4259	9	5	55	Pass
0.4297	9	5	55	Pass
0.4335	9	4	44	Pass
0.4373	8	4	50	Pass
0.4411	8	3	37	Pass
0.4449	7	3	42	Pass
0.4487	7	3	42	Pass

0.4525	5	2	40	Pass	
0.4563	5	2	40	Pass	
0.4601	5	2	40	Pass	
0.4639	5	1	20	Pass	
0.4676	4	1	25	Pass	
0.4714	4	1	25	Pass	
0.4752	4	1	25	Pass	
0.4790	4	1	25	Pass	
0.4828	4	1	25	Pass	
0.4866	4	1	25	Pass	
0.4904	3	1	33	Pass	
0.4942	3	1	33	Pass	
0.4980	3	1	33	Pass	
0.5018	3	1	33	Pass	
0.5056	3	0	0	Pass	
0.5094	3	0	0	Pass	
0.5132	2	0	0	Pass	

Water Quality BMP Flow and Volume for POC #1 On-line facility volume: 0.1314 acre-feet On-line facility target flow: 0.1492 cfs. Adjusted for 15 min: 0.1661 cfs. Off-line facility target flow: 0.0853 cfs.

Adjusted for 15 min: 0.095 cfs.

LID Report

LID Technique		Used for	Total Volumn	Volumn	Infiltration	Cumulative
Percent	Water Quality	Percent	Comment			
		Treatment?	Needs	Through	Volumn	Volumn
Volumn		Water Quality				
			Treatment	Facility	(ac-ft.)	Infiltration
Infiltrated	f	Treated				
			(ac-ft)	(ac-ft)		Credit
			·	•		•

Perlnd and Implnd Changes

No changes have been made.

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